DESIGN OF A MUNICIPAL BATHING BEACH

BY

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THE DESIGN OF A MUNICIPAL BATHING BEACH

A THESIS

PRESENTED BY

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APPROVED:

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FOREWORD

While the primary object of this thesis is for the degree of Bachelor of Science in Civil Engineering, many thanks are extended to Professor J. C. Penn in suggesting "The Design of a Municipal Bathing Beach", from its very inception, which was the chief source of inspiration to the author in conceiving and preparing this thesis for the Armour Institute of Technology.

W. A. NUSSER.

Chicago, Illinois, May, 1917.

INTRODUCTION.

In presenting this work it may not be improper to state the reasons which induced its undertaking, which is to solve one of Chicago's many problems of recreation and health of its citizens in a preliminary design for a centrally located Municipal Bathing Beach and Social Center. The intent of this work is to present in condensed form, everything essential by the most progressive and modern methods. Architectural features and detailed designs have not been entered into.

Believing that the language of a book of this kind should be understood rather than admired, no attempt has been made to use affluent english, but as simple and understandable language as the technical character of the subject would permit.

In preparing the body of this work, many valuable suggestions have been received, which have been incorporated herewith, and which will have contributed to any success the book may attain.

The book is now respectfully submitted, but not without consciousness of its many imperfections, yet with the hope that it may serve as a stepping stone to a more thorough and comprehensive design.

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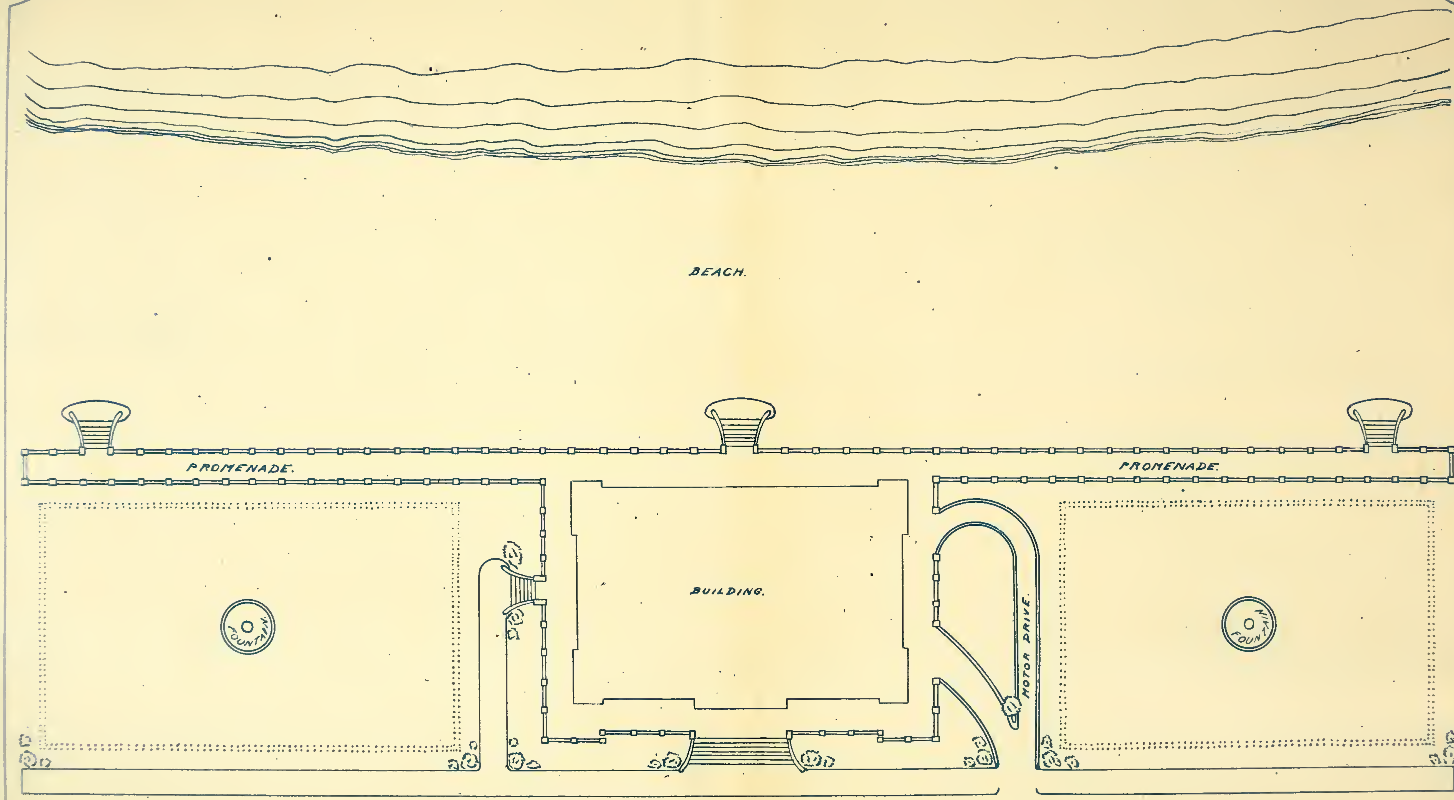


PLAN OF BATHING BEACH.

SCALE 0 12 25 50 75 100 FEET.

PRELIMINARY.

The type of building designed is classified in the Revised Building Ordinances as Class lV. b, and defined as:--

> a building having a parish hall, lodge hall, dance hall, banquet hall, skating rink, assembly hall, halls used for the purpose of exposition and exhibition, or having a hall for theatrical purposes.

The building will have a centralized location and will contain a number of the halls mentioned above, on its upper floors, with an up to date power plant, laundry, gymnasiums, and a swimming tank conveniently located in the basement.

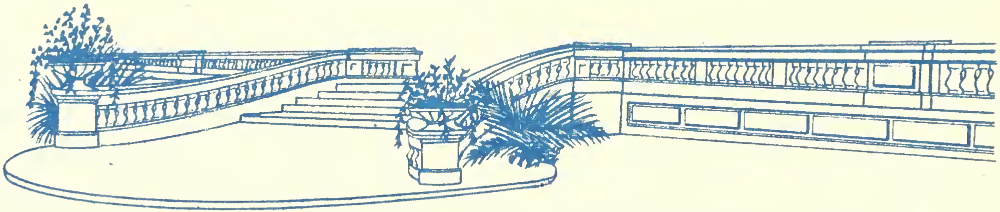
A retaining wall terrace walk and balustrade, beyond which will be sunken Italian Gardens, will surround the front and

sides of the building, and along the lake side or rear of the building will extend a balustraded promenade, containing a six foot, reinforced, concrete, prism walk in the center, below which will be located the locker rooms, showers, comfort stations, etc.

Leading to the side of the building having the dance hall will be a motor drive.

The design for the building, which will be of fireproof construction, - steel skeleton with reinforced concrete for walls, balustrade, promenade, etc., - consists of necessary plans and sketches, which comply with the general and special requirements of The Revised Building Ordinances of the City of Chicago.

The story heights of the first and second floors will be sixteen feet, and that of the basement, ten feet.



VIEW OF STEPS & SECTION OF BALUSTRADE.

PART I.

DESIGN OF STEEL SKELETON CONSTRUCTION.

STAIRWAYS.

In computing the seating capacity of a section of a building of this class, in which the seats are not fixed, an allowance of six square feet of floor area shall be made for each person, and all space between the walls or partitions of such room shall be measured in the computation.

The floor area of a second floor section is (8 x 26 x 28) + (2 x 18 x 28) + (2 x 26 x 14)+ (2 x 18 x 30) + (14 x 18) = 4524 sq. ft. Therefore the seating capacity is 4524 ÷ 6 or 754.

The Building Ordinance requires that halls or rooms having a seating capacity of not more than 900 may be located on any floor and shall have access to at least two interior stairways and not less than one stairway fire escape, the combined width of which shall be equal to at least 18 inches for each one hundred persons for whom accommodations are

provided in said banquet hall or ball room. The combined width required will be $18 \times 7\frac{1}{2} = 135$ in. or $11\frac{1}{4}$ ft.

Stairways at the side of the building will have a width of 12 ft. and those in the front part of the building will have a width of 14 ft. A 3 ft. stairway fire escape will be constructed on the rear face of the building.

The size of tread may be obtained from the following formula:

$$2\ R + T = 24,$$

where R is the rise, which will be made 6 in. and T. the tread, which when computed, will be 12 in. The horizontal projection of the stairway and a 5 ft. landing running in the direction of the stairway is $(12 \times 16 \div 12) + 5 = 21$ ft. and the horizontal projection of the basement stairway is $(12 \times 10 \div 12) + 5 = 15$ ft.

Stacks

The effective area of stack is

$$E = \frac{0.3H}{\sqrt{h}}$$

where H, is the horsepower of the power plant and h is the height of the stack in feet.

It is assumed that H is 100 HP., and that the height of the stack above the roof is 10 ft., thereby making

$$h = 42 + 10 = 52 \text{ ft.}$$

therefore the effective area is

$$E = \frac{0.3 \times 100}{\sqrt{52}} = 4.16$$

The diameter of the stack in inches will be

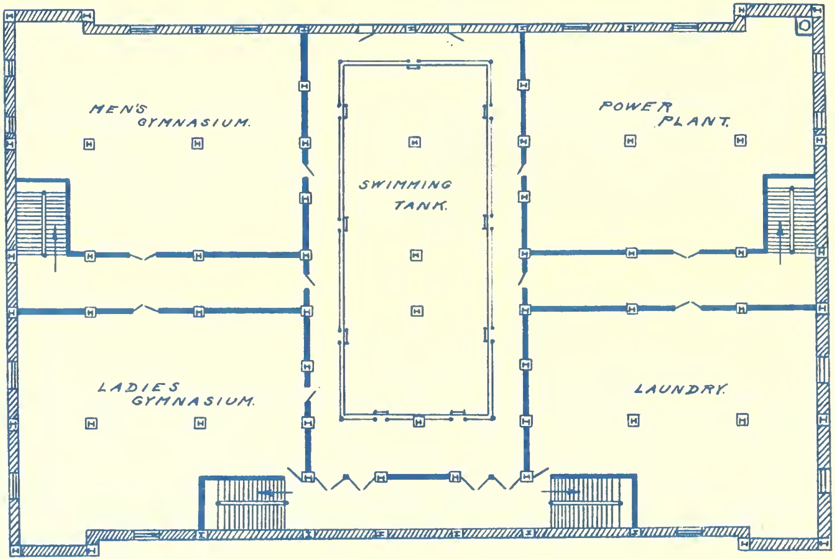
$$13.54 \sqrt{E} + 4'' = 13.54 \sqrt{4.16} + 4'' = 31.6''$$

Windows & Doors.

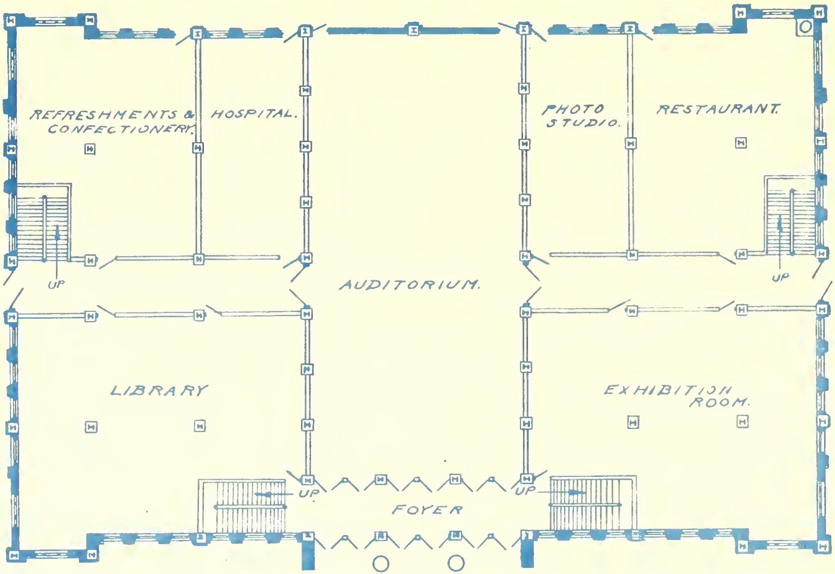
The size of the window openings will be made 5 ft. 8 in. by 8 ft. on all sides and the corner windows will be made 10 ft. by 8 ft.

The Building Ordinance states that the

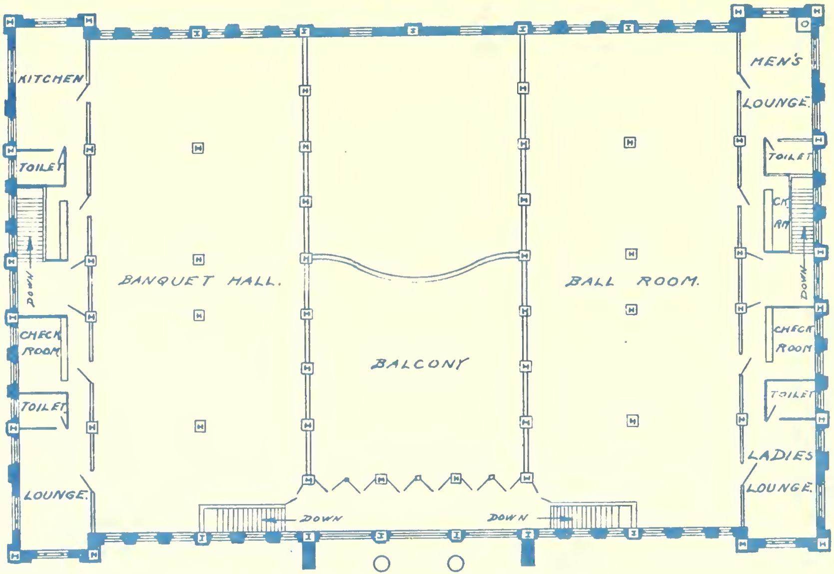
width of corridors, passageways, hallways, and doors adjacent to, connected with, or a part of an auditorium is computed in the same manner as for stairways, and that no such corridor or passageway shall be less than four feet, and no such doorway to be less than three feet in width. The width required as calculated for stairways is $11\frac{1}{4}$ ft. Corridors and passageways will be made 14 ft., and the doors leading thereto will be made 12 ft. The width of the main entrance double doors will be three feet each, thereby making an opening of 6 ft. All other doors will have a width of 4 ft., except for lavatories, which will be 3 ft.



BASEMENT PLAN.
SCALE $\frac{1}{16}'' = 2'\text{-}0''$



FIRST FLOOR PLAN.
SCALE $\frac{1}{16}$" = 2'-0"



SECOND FLOOR PLAN.
SCALE $\frac{1}{16}$" = 2'-0"

DESIGN OF ROOF TRUSS.

A Fink truss will be used to support the roof of the auditorium owing to the short span of 52 ft., and which is more economical than a hinged arch.

The trusses will span 52 ft. and will support a roof having one-fourth pitch, or a rise of 13 ft. Distance between trusses, which is found to be economical as compared to the length of truss, is about one-third, thereby making the distance 14 ft. between centers.

Steel purlins spaced 5 ft. center to center will extend between trusses and will be designed to carry the dead and live loads extending over a purlin panel.

The loads to be carried by the trusses are, a snow load of 15 lbs. per sq. ft. of projected area, a wind load of 20 lbs. per sq. ft. of normal area, a load due to the

trusses' own weight, and a roof consisting of a $2\frac{1}{2}$ in. concrete slab on patent expanded metal, waterproofing of 3 lbs. per sq. ft., steel of 1 lb. per sq. ft. and a $\frac{1}{2}$ in. layer of plaster of 6 lbs. per sq. ft.

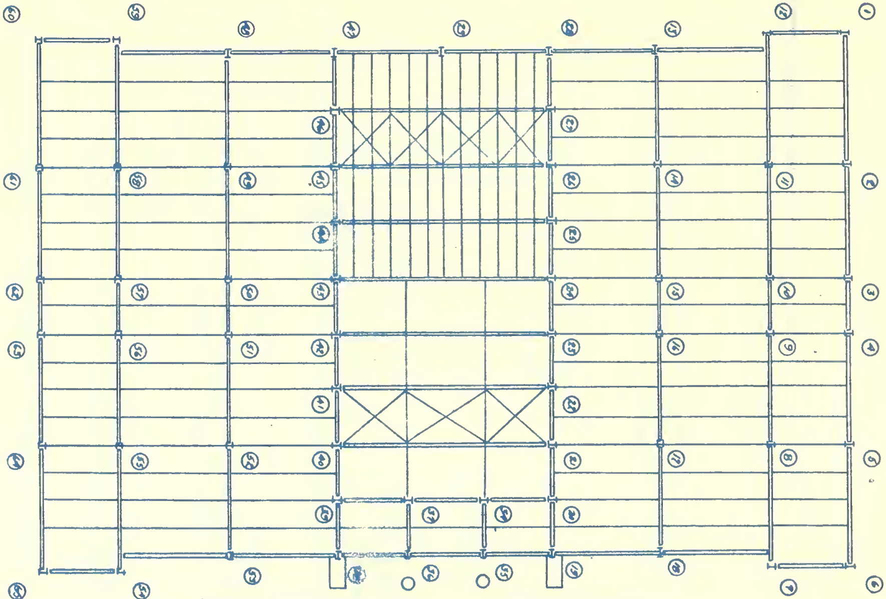
The weight of the trusses for a preliminary design for a span, L, and a roof load of about 40 lbs. per sq. ft. may be obtained by the following formula:

$$W = \frac{65}{40} \times \frac{1}{5} \left(\sqrt{L} + \frac{1}{8} L\right)$$

$$= \frac{65}{40} \times \frac{1}{5} \left(\sqrt{52} + \frac{1}{8} \times 52\right)$$

$$= 4.44 \text{ lbs. / horiz. sq. ft.}$$

Specifications to be used in the design will be those for Structural Work of Buildings, 1915 edition.



TRUSS & GIRDERS. JOISTS & PURLINS.

ROOF FRAMING PLAN.

SCALE $\frac{1}{16}$" = 2'-0"

Design of Purlins.

Loads on purlins, per sq. ft. of roof surface are -

Wind	20#
Waterproofing	3#
Concrete	30#
Plaster	6#
Steel	1#
Total	60#

Secant of the angle of inclination = 1.12

Horizontal projection	=
60 x 1.12	= 67.2# / sq. ft.
Snow load per sq. ft.	
of horizontal proj.	= 15.0# " " "
Purlins' weight	= 2.25#" " "
Total	= 84.45#/ sq. ft.

Roof supported by one intermediate purlin covers (26 ÷ 5) x 14 = 72.8 sq. ft.

Total load on one purlin is 72.8 x 84.45 = 6150#

$$M = \frac{Wl}{8} = \frac{6150 \times 14 \times 12}{8}$$

$$= 129{,}300 \text{ inch pounds.}$$

$$Q = \frac{M}{F} \quad \text{for section modulus.}$$

$$\therefore \quad Q = \frac{129{,}300}{16000} = 8.08$$

Allowed stress per sq. in. equals 16,000 lbs.

Least depth to be not less than one-thirtieth of span: $14 \div 30 = 0.5$ ft. or 6" minimum depth.

Minimum thickness of metal is $\frac{1}{4}$".

Purlin to be used:-

Depth of channel = 9 inches.

Weight per ft. = 13.25#

Moment of Inertia.

$$I_x' = I_x \cos^2 B - I_y \sin^2 B$$

$$= 47.3 \cos^2 26^\circ 34' - 1.8 \sin^2 26^\circ 34'$$

$$= 37.254$$

I_x and I_y are the principal moments of inertia and B is the angle of inclination.

$$S = \frac{I'_x}{c \cos B} = \frac{37.254}{4.5 \times .8944} = 10 \qquad \therefore \text{ O. K.}$$

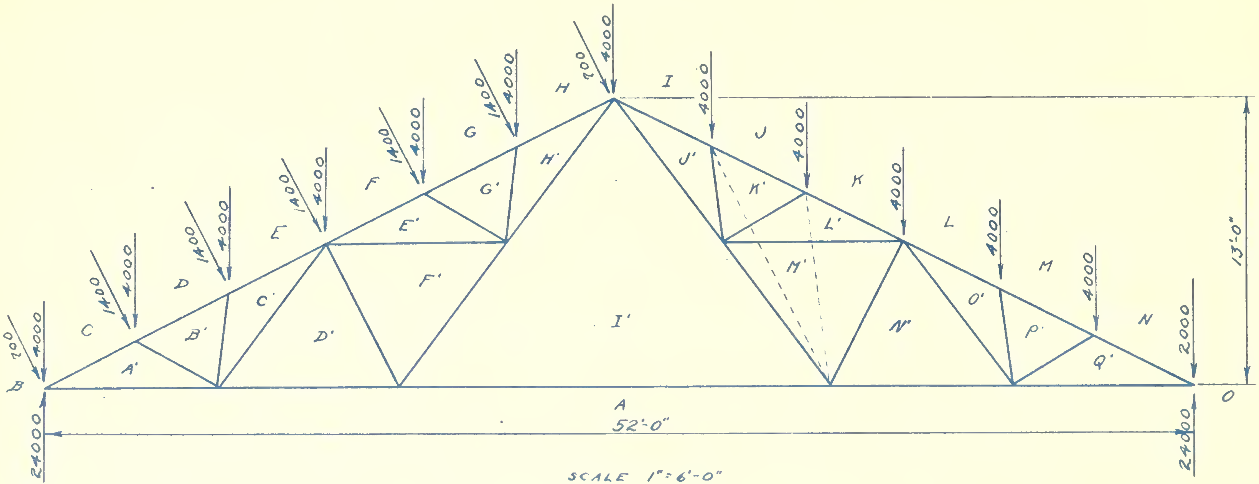


DIAGRAM OF FINK TRUSS.

Stresses in Truss.

The stresses in the members of the truss due to the steady load and those due to the normal wind load of 20 lbs. per sq. ft. of roof surface are given in the following table:

Table of Stresses in Truss.

Member	Steady Load	Wind Load	Maximum.
N Q'	− 49600	− 10100	− 59700
Q'A	+ 44500	+ 8600	+ 53100
M P'	− 45500	− 9200	− 54700
P'Q'	− 4300	− 1700	− 6000
L O'	− 46100	− 10100	− 56200
O'P'	− 4250	− 1700	− 5950
O'N'	+ 8000	+ 3150	+ 11150
N'A	+ 36500	+ 5600	+ 42100
K L'	− 44250	− 10050	− 54300
L'M'	+ 8000	+ 3100	+ 11100
M'N'	− 10800	− 4200	− 15000
M'I'	+ 12100	+ 4650	+ 16750
I'A	+ 24400	+ 900	+ 25300
J K'	− 40000	− 9150	− 49150
K'L'	− 4300	− 1650	− 5950
I J'	− 40500	− 10075	− 50575
J'K'	− 4300	− 1720	− 6020
J'I'	+ 20100	+ 7800	+ 27900
H H'	− 48600	− 10175	− 50675
H'I'	+ 20100	+ 6850	+ 26950
G G'	− 40000	− 9250	− 49250
G'H'	− 4300	− 1700	− 6000
F E'	− 44250	− 10150	− 54400
E'G'	− 4300	− 1650	− 5950
E'F'	+ 8000	+ 3100	+ 11100
F'I'	+ 12100	+ 4700	+ 16800
E C'	− 46100	− 10100	− 56200
C'D'	+ 8000	+ 3100	+ 11100
D'F'	− 10800	− 4200	− 15000

Table of Stresses in Truss - Continued

Member	Steady Load	Wind Load	Maximum
D'A	+ 36500	+ 9400	+ 45900
D B'	− 45500	− 9175	− 54675
B'C'	− 4250	− 1700	− 5950
B'A'	− 4300	− 1700	− 6000
A'A	+ 44500	+ 12500	+ 57000
C A'	− 49600	− 10125	− 59725

Design of Tension Members.

Specifications allows 16000 lbs. per sq. in. as the allowed stress on the net sections of tension members and requires that rivet holes be 1/8 in. larger than the diameter of rivet shall be taken out in determining the net section of these members. The size of rivets used will be $\frac{3}{4}$ in. and one rivet hole will be counted out of each angle.

Members: A A', D'A, Q'A, N'A.

Stress = 57,000 lbs.

Required net area = 57,000 ÷ 16,000 = 3.56 sq. in.

Area of 2 angles $3\frac{1}{2}$" x $2\frac{1}{2}$" x 3/8" = 4.22 " "

Area of rivet holes = 2 x .28 = .56 sq. in.

Net section = 4.22 - 0.56 = 3.66 sq. in.

Strength of members = 3.66 x 16000 = 58500 #

Members: C'D', O'N', E'F', L'M'.

Stress = 11,100 lbs.

Required net area = 11100 ÷ 16000 = 0.695 sq. in.

Area of one angle $2\frac{1}{2}$" x $2\frac{1}{2}$" x $\frac{1}{4}$" = 1.19 sq. in.

Area of rivet hole = 0.22

Net section = 1.19 - 0.22 = 0.97 sq. in.

Strength of members = 0.97 x 16000 = 15500#

Members: H'I', I'J', F'I', M'I'.

Stress = 27,700 lbs.

Required net area = 27700 ÷ 16000 = 1.73 sq. in.

Area of 2 angles $2\frac{1}{2}$" x $2\frac{1}{2}$" x $\frac{1}{4}$" = 2.38 sq. in.

Area of rivet holes = 2 x 0.22 = 0.44 sq. in.

Net section = 2.38 - 0.44 = 1.94 sq. in.

Strength of members = 1.94 x 16000 = 31,000#

Member I'A.

Stress = 29,100 lbs.

Required net area = 29100 ÷ 16000 = 1.82 sq. in.

Area of 2 angles $3\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{1}{4}" = 2.88$ sq. in.

Area of rivet holes $= 2 \times 0.22 = 0.44$ sq. in.

Net section $= 2.88 - 0.44 = 2.44$ sq. in.

Strength of member $= 2.44 \times 16000 = 39040\#$

Design of Compression Members.

The specification allows stress per square inch to be found by the formula $16000 - 70\,\frac{l}{r}$, where l is the effective length of the member in inches, and r is the radius of gyration of the member in inches.

Members: C A', D B', E C', F E', G G',
H H', I J', J K', K L', L O',
M P', N Q'.

Stress $= -59500$ lbs.

Length of members are 58.136 in.

Radius of gyration of 2 angles, $4" \times 3" \times 3/8"$ is 1.12 in.

Allowed stress per sq. in. is

$$16000 - 70\,\frac{58.136}{1.12} = 12770 \text{ lbs.}$$

Required area = 59500 ÷ 12770 = 4.58 sq. in.

Area of the two angles is 4.96 sq. in.

Strength = 4.96 x 12770 = 63,340 lbs.

For axial compression of gross sections, for ratio of l/r up to 120, the allowed stress per square inch is given by the formula 19000 − 100 l/r, with a maximum stress of 13,000 lbs.

Members: A'B', B'C', E'G', G'H', O'P', P'O', L'K', K'J'.

Stress = − 6000 lbs.

Length of member is 52.4 in.

Radius of gyration of angle $2\frac{1}{2}"$ x 2" x $\frac{1}{4}"$ is 0.78 in.

Allowed stress per sq. in. is

$$19000 - 100\,\frac{52.4}{0.78} = 12,280 \text{ lbs.}$$

Required area is 6000 ÷ 12280 = 0.5 sq. in.

Area of angle $2\frac{1}{2}"$ x 2" x $\frac{1}{4}"$ is 1.06 sq. in.

Strength = 1.06 x 12280 = 13,000 lbs.

Members: D'F', N'M'.

Stress = − 15,000 lbs.

Length of members are 87.2 in.

Radius of gyration of 2 angles $2\frac{1}{2}$" x 2" x $\frac{1}{4}$"

is 0.78 in.

Allowed stress per sq. in. is

$$19000 - 100\ \frac{87.2}{0.78} = 8000 \text{ lbs.}$$

Required area is 15000 ÷ 8000 = 2.00 sq. in.

Area of angles $2\frac{1}{2}$" x 2" x $\frac{1}{4}$" is 2.12 sq. in.

Strength = 2.12 x 8000 = 17,000 lbs.

In the above design, the angles $2\frac{1}{2}$" x 2" x $\frac{1}{4}$" are the minimum size which are permissible.

DESIGN OF JOISTS.

Roof Joists

Length of joists = 26'- 0" * Loads on joists

15" terra cotta	42# / sq. ft.
concrete	30# " " "
ceiling	5# " " "
roofing	6# " " "
Live load	25# " " "
Total =	108# " " "

$$\frac{M}{s} = \frac{I}{c} = \frac{1}{8} \times \frac{108 \times 7 \times 26^2 \times 12}{16000}$$

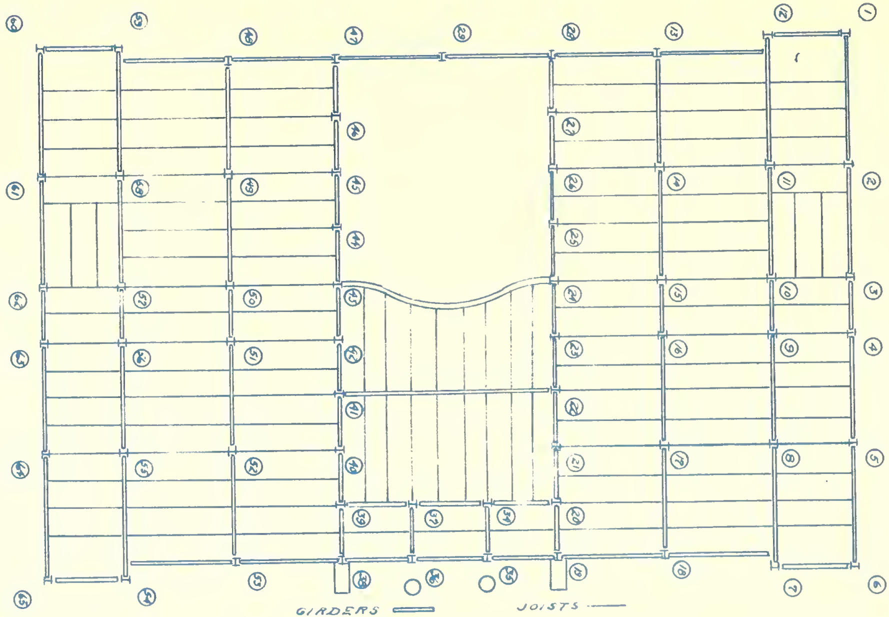
$$= 47.7 \text{ in.}^3$$

Assume a 15" - 36 lb. I. Beam

$$\frac{M}{s} = \frac{I}{c} = \frac{1}{8} \times \frac{36 \times 26^2 \times 12}{16000}$$

$$= 2.3 \text{ in.}^3$$

Therefore all roof joists will be 15" - 36 lb. I. Beams, spaced 7 ft. C. to C.



SECOND FLOOR FRAMING PLAN.

SCALE $\frac{1}{16}'' = 2'\text{-}0''$

Second Floor Joists 11 - 14

Length of joists = 26' - 0"	Loads on joists
15" terra cotta	42# / sq. ft.
concrete	30# " " "
ceiling	5# " " "
flooring	4# " " "
steel	12# " " "
Live load	100# " " "
Total =	193# " " "

$$\frac{M}{s} = \frac{I}{c} = \frac{1}{8} \times \frac{193 \times 7 \times 26^2 \times 12}{16000}$$

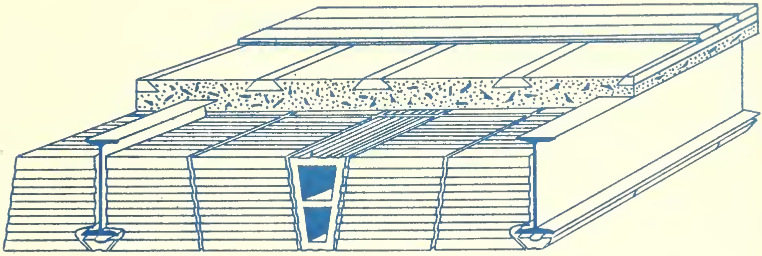
$$= 86.2 \text{ in.}^3$$

Use 15" - 70# I. Beams; spaced 7ft. C. to C.

Second Floor Joists 2 - 11

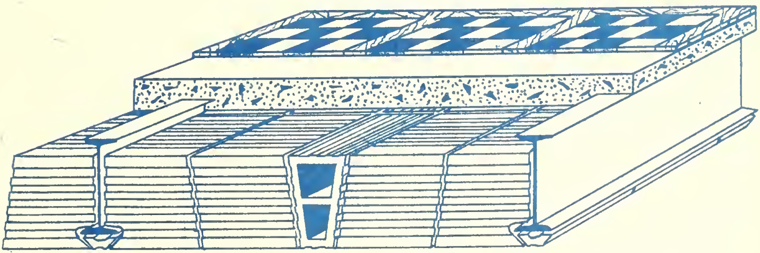
Length of joists = 18' - 0"	Loads on joists
15" terra cotta	42# / sq. ft.
concrete	30# " " "
ceiling	5# " " "
flooring	4# " " "
partition	20# " " "
steel	12# " " "
Live load	100# " " "
Total =	213# " " "

TYPICAL FLOOR DESIGN



WOOD FLOOR ON TOP OF CONCRETE FILL.

FIRST FLOOR DESIGN



MARBLE & MOSAIC FLOOR.

$$\frac{M}{s} = \frac{I}{c} = \frac{1}{8} \times \frac{213 \times 7 \times 18^2 \times 12}{16000}$$

$$= 45.3 \text{ in.}^3$$

Use 15" - 36# I. Beams; spaced 7ft. C. to C.

Second Floor Joists 20 - 34

Length of span = 17.33'	Load on joists
15" terra cotta	42# / sq. ft.
concrete	30# " " "
ceiling	5# " " "
flooring	10# " " "
steel	12# " " "
Live load	100# " " "
Total =	200# " " "

$$\frac{M}{s} = \frac{I}{c} = \frac{1}{8} \times \frac{200 \times 7 \times 17.33^2 \times 12}{16000}$$

$$= 39.4 \text{ in.}^3$$

Use 15" - 36# I. Beams, spaced 7' - 0" C. to C.

Balcony Joists.

The lengths of the balcony joists are 28 ft. and spaced 5.77 ft. center to center. The load carried by these joists is similar to

that carried by joists 11 - 14, which is 193 lbs. per sq. ft.

$$\frac{M}{s} = \frac{I}{c} = \frac{1}{8} \times \frac{193 \times 5.77 \times 28^2 \times 12}{16000}$$

$$= 82 \text{ in.}^3$$

Use 15" - 65# I. Beams, spaced 5.77 ft. C. to C.

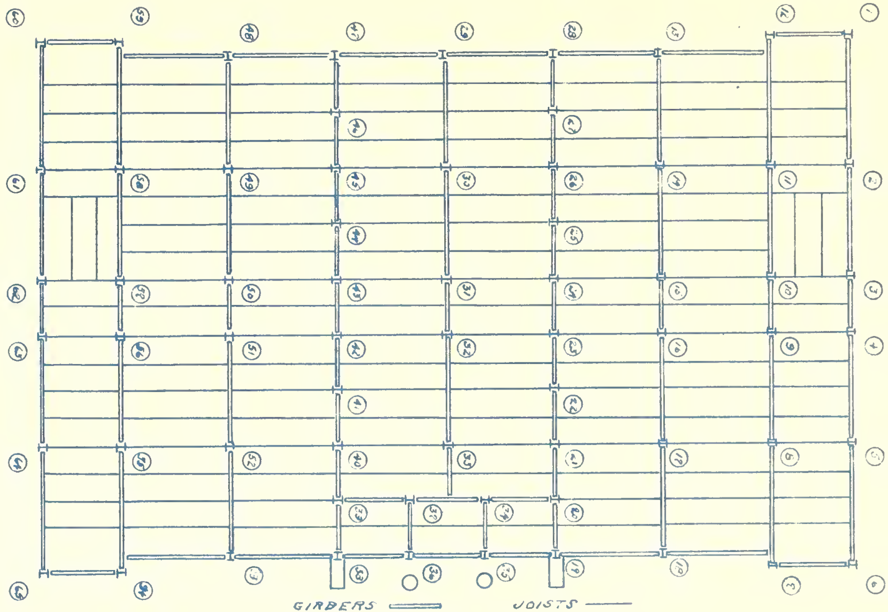
First Floor Joists 11 - 14

Length of span = 26' - 0"	Load on joists
15" terra cotta	42# / sq. ft.
concrete	30# " " "
ceiling	5# " " "
flooring	10# " " "
steel	12# " " "
partition	20# " " "
Live load	100# " " "
Total =	219# " " "

$$\frac{M}{s} = \frac{I}{c} = \frac{1}{8} \times \frac{219 \times 7 \times 26^2 \times 12}{16000}$$

$$= 97.1 \text{ in.}^3$$

Use 18" - 65# I. Beams, spaced 7' - 0" C. to C.



FIRST FLOOR FRAMING PLAN.

SCALE $\frac{1}{16}'' = 2'\text{-}0''$

First Floor Joists 2 - 11

Length of span = 18' - 0"   Loads on joists

15" terra cotta	42# / sq. ft.
concrete	30# " " "
ceiling	5# " " "
flooring	10# " " "
steel	12# " " "
partitions	20# " " "
Live load	100# " " "
Total =	219# " " "

$$\frac{M}{s} = \frac{I}{c} = \frac{1}{8} \times \frac{219 \times 7 \times 18^2 \times 12}{16000}$$

$$= 46.6 \text{ in.}^3$$

Use 15" - 36# I. Beams, spaced 7'- 0" C. to C.

First Floor Joists 20 - 34

These joists will carry a load equal to that on the second floor, and therefore 15" - 36# I. Beams will be used.

Tie Rods.

Owing to the side thrusts on the joists, produced by the tapered faces and the central

blocks of terra cotta, it is found necessary to counterbalance these thrusts by means of tie rods in order to prevent sidewise deflection. The total thrust of arch, the net area of tie rods required, the maximum distance between the tie rods, and the combined fiber stress is found as follows:

Floor panel 26 ft. by 7 ft., made of 15 in. flat terra cotta blocks, is to support a uniform dead and live load of 210 lbs. / sq. ft.

Entire floor load is 26 x 7 x 210 = 38220 lbs. Size of I. Beam is 15" - 70# and assume 7/8" tie rods.

Thrust of arch per lin. ft.,

$$p = \frac{3 \times 210 \times 7 \times 7}{2\ (15 - 2.4)} = 1225 \text{ lbs.}$$

Total thrust of arch,

$$P = \frac{3 \times 210 \times 7 \times 7 \times 26}{2\ (15 - 2.4)} = 31850 \text{ lbs.}$$

Total arear of tie rods,

$$A = \frac{210 \times 7 \times 7 \times 26}{10667\ (15 - 2.4)} = 1.99 \text{ sq. in.}$$

Maximum spacing of tie rods,

$$L_s = \frac{10667 \times 0.42 \times (15 - 2.4)}{210 \times 6 \times 6} = 5.4 \text{ ft.}$$

Bending Moment, vertical loading,

$$M_{1-1} = \frac{7 \times 26 \times 210 \times 26 \times 12}{8 \times 12} = 745300\text{" lbs.}$$

Bending Moment, horizontal thrust,

$$M_{2-2} = \frac{1225 \times 5.4 \times 5.4 \times 12}{12} = 35770\text{" lbs.}$$

Combined fiber stress,

$$f = \frac{745300}{88.5} + \frac{35770}{9.4} = 8800 \text{ lbs. / sq. in.}$$

When tie rods are used they will be placed in the line of thrust and about 5 ft. C. to C.

SPANDREL GIRDERS.

The building ordinances of the City of Chicago require the enclosing walls in buildings of steel skeleton construction, which do not carry the weight of floors or roof, shall be not less than twelve inches in thickness, provided such walls be thoroughly anchored to the steel skeleton. Whenever the weight of such walls rests on beams or columns, such beams or columns shall be made strong enough in each story to carry the weight of the wall resting upon them, without reliance upon the walls below them. All walls shall be of fireproof or incombustible material. Therefore a fourteen inch wall will be used composed of 6 in. finished terra cotta - surface of 22 lbs. per sq. ft., 8 in. of common brick of 80 lbs. per sq. ft., and a layer of plaster of 5 lbs. per sq. ft., thereby making a total weight of 107 lbs. per

sq. ft. of wall area.

Above the roof level of the building will be a four foot parapet wall and balustrade constructed of the same material as that used for the walls.

The size of the windows will be 5 ft. 8 in. by 8 ft. for the sides and ends of the building, while those at the corners of the building will be 10 ft. by 8 ft. Eight feet being the height in each case.

The following spandrel girders are designed so that they may serve for any story of the building.

Spandrel Girder 28 – 29

Thickness of wall will be 14 in.

6" finished terra cotta	= 22# / sq. ft.
8" common brick	= 80# / " "
plaster	= 5# / " "
	107# / sq. ft.

Length of span is 26 ft. - height of story 16 ft.

Area of wall = 16' x 26' = 416 sq. ft.

" " door = 4' x 7' = 28 " "

Total area = 388 sq. ft.

Weight of wall = 107 x 388 = 41,516#

Weight " floor 207 x 26 x $3\frac{1}{2}$ = 18,837#

Total load = 60353 lbs.

$$\frac{M}{s} = \frac{I}{c} = \frac{60353 \times 26 \times 12}{8 \times 16000} = 156.5$$

Assume a 70# girder.

B. M. due to weight of girder

= 1/8 x 70 x 676 x 12 = 68,000 in. lbs.

$$\frac{M}{s} = \frac{I}{c} = \frac{68,000}{16,000} = 4.2$$

Total section modulus is 160.7

Therefore use 24" - 69.5# I. Beam, whose section modulus is 160.7 in.^{3}.

Spandrel Girder 12 - 13

Span is 26 ft.

Windows will be 5' - 8" x 8' - 0"

Area of wall = 16' x 26' = 416 sq. ft.

Area of 3 windows 5' - 8" x 8' - 0" = 136.08 sq. ft.

Therefore the total wall area equals

416 - 136 = 280 sq. ft.

Weight of floor = 207 x 26 x 3½ = 18,837#

" " wall = 107 x 280 = 29,960#

Assume a 70# I. Beam - wt. = 70 x 26

= 1820#

Therefore R = 25,310 lbs.

Total load = 29,960 + 18,837 + 1,820 = 50,617#

$$\frac{M}{s} = \frac{I}{c} = \frac{50617 \times 26 \times 12}{8 \times 16000} = 123.3$$

Therefore use 24" - 69.5# I. Beam, whose section modulus is 160.7 in.^{3}.

Spandrel Girder 1 - 12

Span is 18 ft.

Window will be 8' - 0" x 10' - 0"

Area of wall = 16' x 18' = 288 sq. ft.

Area of window 8' x 10' = 80 sq. ft.

Total area of wall = 208 sq. ft.

Weight of wall = 107 x 208 = 22,256#

" " floor = 207 x 18 x $6\frac{1}{2}$ = 24,219#

Assume weight of I. Beam as 50# / lin. ft.

Total load = 22256 + 24219 + 900 = 47,375#

$$\frac{M}{s} = \frac{I}{c} = \frac{47375 \times 18 \times 12}{8 \times 16000} = 80.0$$

Therefore use an 18"- 46# I. Beam, whose section modulus is 81.5 in.^{3}.

Spandrel Girder 1 - 2

Span is 31 ft.

Reaction from the 15" - 36# I. Beams

$\frac{1}{2}$ x 26,730 = 13,365#

Windows 10' - 0" x 8' - 0"
5' - 8" x 8' - 0"

∴ area = 80 + 45.4 = 125.4 sq. ft.

Area of wall = 16 x 31 = 496 sq. ft.

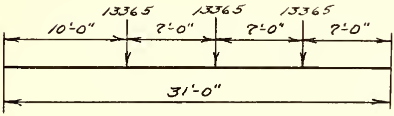
Total area of wall = 496 - 125.4 = 370.6 sq. ft.

Weight of wall $= 107 \times 370.6 = 39,654\#$

Assume a 100# I. Beam.

Weight of I. Beam $= 100 \times 31 = 3100\#$

$\therefore$ total uniform load $= 42,754\#$



Reaction R_2 due to conc. load =

$$R_2 = \frac{13365 \times 10 + 13365 \times 17 + 13365 \times 24}{31}$$

$$= \frac{681615}{31} = 22,000 \text{ lbs.}$$

$\therefore$ R $= 18,095$ lbs.

B. M. due to uniform load $= 1/8 \times 42754 \times 31 \times 12$

$= 2,138,060$ lbs.

B. M. due to conc. load $= (22000 \times 14 - 13365 \times 7)$

$\times 12 \quad = 2,573,340$ lbs.

Total B. M. $= 4,711,400$ in. lbs.

$$\frac{M}{s} = \frac{I}{c} = \frac{4711400}{16000} = 294.5$$

Therefore use a riveted plate girder having a web plate 27" x 5/16", flange angles 5" x $3\frac{1}{2}$" x 3/8" and two plates 12" x 3/8", whose section modulus is 304.5 in.^{3}.

Spandrel Girder 2 - 3.

Span 28 ft. Reaction from joists = 13365#

Area of wall 16' x 28' = 448 sq. ft.

Area of 3 windows 5' - 8" x 8' - 0" = 136 sq. ft.

∴ total wall area = 448 - 136 = 312 sq. ft.

Weight of wall = 107 x 312 = 33384 lbs.

Assume weight of girder as 90 # / lin. ft.

Total weight = 2520#

B. M. due to concentrated load =

(20047 x 14 - 13365 x 7) 12 = 2,245,240"#

B. M. due to uniform load..... =

1/8 x 35904 x 28 x 12 = 1,520470"#

Total M = 3,765,710"#

$$\frac{M}{s} = \frac{I}{c} = \frac{3765710}{16000} = 235.3$$

Therefore use a riveted plate girder having a web plate 27" x 3/8" and flange angles 5" x $3\frac{1}{2}$" x $\frac{1}{2}$", whose section modulus is 237.8 in.^{3}.

Spandrel Girder 3 - 4.

Span is 14 ft.

Area of wall 16 x 14 = 224 sq. ft.

" " 2 windows 5' - 8" x 8' - 0" = 90.72 sq. ft.

Wall area = 224 - 90.72 = 133.3 sq. ft.

Weight of wall = 107 x 133.3 = 14,263#

Reaction due to joist = 13,365#

Assume weight of girder as 40# / lin. ft.

Total weight = 560#

B. M. due to uniform load =

1/8 x 14823 x 14 x 12 = 311280 in. lbs.

B. M. due to concentrated load at center of span =

1/4 x 13365 x 14 x 12 = 561330 in. lbs.

Total M = 872610 in. lbs.

$$\frac{M}{s} = \frac{I}{c} = \frac{872610}{16000} = 54.4$$

∴ use 15" - 42# I. Beam, whose section modulus is 54.0 in.^{3}.

DESIGN OF GIRDERS.

Roof Girders

Girder 13 – 14

The reaction due to a joist is 20592 lbs., which is concentrated at points 7 ft. center to center. The bending moment due to the concentrated loads is (30888 x 14 – 20592 x 7) 12 = 3459460 in. lbs. Therefore the section modulus equals 3459460 ÷ 16000 or 215.1 $in.^3$. Assuming a 90 lb. built up girder, the bending moment of the girder will equal 1/8 x 90 x 784 x 12 = 105840 in. lbs. The section modulus will then be 105840 ÷ 16000 = 6.6, giving a total section modulus of 221.7 $in.^3$.

A riveted plate girder will be used, having a web plate 26" x 3/8" and four angles 5" x 3½" x ½".

Girder 11 – 10

The reaction due to the 18 ft. joists is

$\frac{1}{2}$ x 14256 = 7128 lbs. and that due to the 26' joists is $\frac{1}{2}$ x 20592 = 10296 lbs., making a concentrated load of 17424 lbs. The bending moment due to the concentrated loads is (26136 x 14 – 17424 x 7) 12 = 2927200 in. lbs. Assume an 80 lb. girder, its bending moment will be 1/8 x 80 x 784 x 12 = 94080 in. lbs. The total moment is 3021480 in. lbs. and the section modulus is 3021480 ÷ 16000 = 188.8 in.^{3}.

The riveted plate girder to be used will have a web plate 26" x 3/8" and four flange angles 4" x 3" x $\frac{1}{2}$" and having a section modulus of 193.5.

Girder 10 – 9

The concentrated load on the girder is 17424 lbs. and the bending moment due to it is $\frac{1}{4}$ x 17424 x 14 x 12 = 731810 in. lbs. Assuming a 40 lb. girder, its bending moment is 1/8 x 40 x 196 x 12 = 11750 lbs. The total bending moment is 743560 in. lbs. and the

section modulus is 743560 ÷ 16000 = 46.5. Therefore a 15" – 36# I. Beam will be used, having a section modulus of 54.0.

Girder 15 – 16

The reaction due to the joists is 20592 lbs., and the bending moment due to this concentrated load is $\frac{1}{4}$ x 20592 x 14 x 12 = 865000 in. lbs. Assuming a 60 lb. girder, its bending moment is 1/8 x 60 x 196 x 12 = 17640 in. lbs. The total bending moment is 882640 in. lbs. and the section modulus is 882640 ÷ 16000 = 55.2. A 21" – 57.5# I. Beam will be used whose section modulus is 116.9.

Girder 24 – 25

The reaction due to the joists is 10296 lbs. and the bending moment due to this concentrated load is $\frac{1}{4}$ x 10296 x 14 x 12 = 432500 in. lbs. Assuming a 40 lb. girder, its bending moment is 1/8 x 40 x 196 x 12 = 11750 in. lbs. The total bending moment is 444250 in. lbs. and the section modulus is

444250 ÷ 16000 = 27.7. The smallest size beam that can be used is a 15", and therefore a 15" - 36# beam will be used.

Girder 11 - 12

The weight due to the concentrated loads from the joists is 17424 lbs., and that due to the spandrel girder 12 - 13 is 25310 lbs. The maximum reaction due to the concentrated loads is [17424 x (7 - 14 - 21) - 25310 x 28] ÷ 31 = 46467 lbs. Assume a 100 lb. girder, the bending moment due to the concentrated loads is (46467 x 17 - 25310 x 14 - 17424 x 7) 12 = 3763600 in. lbs., and that due to the weight of the girder is (1550 x 17 - 100 x 17 x 8.5) 12 = 142800 in. lbs. The total bending moment is 3906400 in. lbs. and the section modulus is 3906400 ÷ 16000 = 244.2. Therefore a riveted plate girder will be used, having a web plate 26" x 5/16" and four flange angles 6" x 4" x ½" and having a section modulus of 252.0 in.^{3}.

SECOND FLOOR GIRDERS

Girder 13 - 14

The reaction due to the 15" - 70# joists and panel load is 1820 + 181 x 7 x 26 = 34742 lbs., which amount is concentrated at points 7 ft. center to center upon a girder whose length is 28 ft. The bending moment due to the concentrated loads is (52113 x 14 - 34742 x 7) 12 = 5836660 in. lbs. Assuming a built up girder of 140 lbs., its bending moment will then be 1/8 x 140 x 784 x 12 = 164800 in. lbs., thereby making a total bending moment of 6001460 in. lbs. Therefore the required section modulus is

$$6001460 \div 16000 = 375.1.$$

A riveted plate girder will be used having a web plate 26" x 3/8", four flange angles 5" x $3\frac{1}{2}$" x $\frac{1}{2}$", and two plates 12" x $\frac{1}{2}$", whose total section modulus is 377.4 $in.^3$.

Girder 10 - 11.

The reaction due to the 15" - 36# joists and its panel load is ½ x 25848 = 12924 lbs. and that due to the 15" - 70# joists and its panel load is ½ x 34742 = 17371 lbs., making a total concentrated load of 12924 + 17371 = 30295 lbs. These loads are concentrated 7 ft. center to center upon a girder whose length is 28 ft. The uniform load will consist of a 4 in. tile partition with two coats of plaster whose weight is 25 lbs. per sq. ft., or 375 lbs. per lin. ft. and the weight of an assumed girder is 130 lbs. per lin. ft., thereby making a total uniform load of 505 lbs. per lin. ft. The bending moment due to the concentrated loads is (45443 x 14 - 30295 x 7) 12 = 5089640 in. lbs. and that due to the uniform load is 1/8 x 505 x 784 x 12 = 593880 in. lbs. making a total bending moment of 5683520 in. lbs. Therefore the required section modulus is 5683520 ÷ 16000 = 355.2. A riveted plate girder will be used having a web plate 26" x 5/16", four

flange angles 5" x $3\frac{1}{2}$" x $\frac{1}{2}$", and two plates 12" x $\frac{1}{2}$" whose total section modulus is 370.7 in.^{3}.

Girder 9- 10

The reaction due to the 15" - 70# joists and its panel load is $\frac{1}{2}$ x 34742 = 17371 lbs. and that due to the 15" - 36# joists and its panel load is $\frac{1}{2}$ x 25848 = 12924 lbs. making a total concentrated load of 30295 lbs. The uniform load will consist of a 4" tile and plaster partition having a weight of 375 lbs. per lin. ft. and the weight of an assumed girder is 60 lbs. per lin. ft. thereby giving a total uniform load of 435 lbs. per lin. ft. The bending moment due to the concentrated load at the center of the 14 ft. span is $\frac{1}{4}$ x 30295 x 14 x 12 = 1272400 in. lbs. and that due to the uniform load is 1/8 x 435 x 196 x 12 = 128000 in. lbs. giving a total bending moment of 1400400 in. lbs. Therefore

the required section modulus is 1400400 ÷ 16000 = 87.5. An 18" - 55# I. Beam whose section modulus is 88.4 in.^{3} will be used.

Girder 15 - 16

For girder 15 - 16, the same size as that used on the first floor will be used, which is a 21" - 57.5# beam, whose section modulus is 116.9 in.^{3}.

Girder 24 - 25

The concentrated load upon the center of the 14 ft. girder is equal to the reaction due to the 15" - 70# joist and its panel load which is $\frac{1}{2}$ x 1820 + 181 x 7 x 26 = 17371 lbs. The uniform load will consist of the weight of the tile partition of 15 x 1 x 25 = 375 lbs. per lin. ft. and the weight of an assumed girder is 60 lbs. per lin. ft. The bending moment due to the concentrated load is $\frac{1}{4}$ x 17371 x 14 x 12 = 978580 in. lbs. and that due to the uniform load is 1/8 x 435 x 196 x 12

= 128000 in. lbs. thereby making a total bending moment of 1106580 in. lbs. The required section modulus is 1106580 ÷ 16000 = 69.2, and a 15" - 60# I. Beam, whose section modulus is 81.2 in.^{3} will be used.

Girder 11 - 12

Owing to the decrease in floor load and an increase in the uniform load it will be found that the same size riveted plate girder as that used for the first floor may be used for girder 11 - 12. This girder consisted of a web plate 26" x 3/8", four flange angles 5" x 3½" x ½", and two plates 12" x 5/8", whose total section modulus is 415.2 in.^{3}.

Girder 20 - 34

The joists in the balcony will be spaced 5.77 ft. center to center and thereby making two concentrated loads upon the girder 20 - 34 whose length will be 17.33 ft. These concentrated loads will be due to the reaction of

the 65 lb. joists and their panel loads, which is $\frac{1}{2}$ (1820 + 193 x 5.77 x 28) = 16506 lbs., and the bending moment due to the concentrated loads is (16506 x 8.67 - 16506 x 2.9) 12 = 1142880 in. lbs. The uniform floor load will equal 3.5 x 17.33 x 1.93 = 11690 lbs., which will give a bending moment of 1/8 x 11690 x 17.33 x 12 = 303885 in. lbs., and the bending moment of an assumed girder of 60 lbs. per lin. ft. is 1/8 x 60 x 299.3 x 12 = 26940 in. lbs. Therefore the total bending moment is 1473705 in. lbs., and the required section modulus is 1473705 ÷ 16000 = 92.1. A 21" - 57.5# beam, whose section modulus is 116.9 in.^{3} will be used.

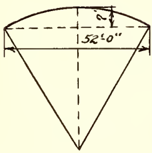
Girder 24 - 43

In the design of the curved balcony girder 24 - 43, it will be figured as a straight beam with concentrated loads due to the joists which are spaced 5.77 ft. center to

center.

The length of the girder is obtained as follows;

$$R^2 = R^2 - 14\ R + 49 + 26^2$$



$$14\ R = 49 + 676 = 725$$

$$R = 51.8 \text{ ft.}$$

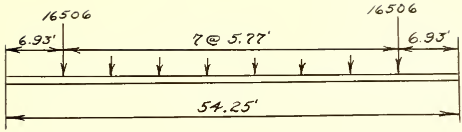
$$\sin\theta = 26 \div 51.8$$

$$= 0.502$$

$$\therefore \theta = 30^{\circ}\ 10'$$

therefore the length of arc is equal to 1/6 of the circumference of a circle, whose radius is 51.8 ft. or 1/6 x 325.5 = 54.25 ft. which will be the length of the girder 24 – 43.

The concentrated loads due to the reactions of the joists and their panel loads, which is 16506 lbs., as found for girder 20 – 34, will be 5.77 ft. center to center and 6.93 ft. from each end as shown in the following figure.



The bending moment due to the concentrated loads is $[66024 \times 27.3 - 16506\ (20.2 + 14.42 + 8.65 + 2.88)]\ 12 = 12400000$ in. lbs. and that due to an assumed, uniform weight of a girder and plaster of 220 lbs. per lin. ft. is $1/8 \times 220 \times 29.43 \times 12 = 971200$ in. lbs.; thereby making a total bending moment of 13,371,200 in. lbs. The required section modulus is $13,371,200 \div 16,000 = 835.7$, and a riveted plate girder having a web plate 36" x 7/16", four flange angles 6" x 4" x 5/8", and two plates 14" x 3/4", whose total section modulus is 840.4 in.^{3} will be used.

Girder 22 - 41

The girder 22 - 41 will span 52 ft. and

will carry the concentrated loads due to the reactions of the joists and their panel loads. Since this girder is a few feet shorter and carries twice as much concentrated load as girder 24 - 43, a section modulus which is twice that which was used for girder 24 - 43 may be used for girder 22 - 41.

FIRST FLOOR GIRDERS.

Girder 13 - 14

The reaction due to the joists and panel load is 39364 lbs., which is concentrated at points 7 ft. center to center. The length of span is 28 ft., and the bending moment due to the concentrated loads is (58046 x 14 - 39364 x7) 12 = 6445150 in. lbs. Assuming a built up girder of 140 lbs., its bending moment will then be 1/8 x 140 x 784 x 12 = 164800 in. lbs. Therefore the total bending moment is 6609950 in. lbs. and the required section modulus is 6609950 ÷ 16000 = 413.1. A riveted plate girder will be used having a web plate 26" x 3/8", four flange angles 5" x 3-1/3" x ½", and two plates 12" x 5/8" whose total section modulus is 415.2 in.^{3}.

Girder 10 - 11

The reaction due to the 18 ft. joists and panel load is ½ x 39364 = 19682 lbs. and that due to the 26 ft. joists and panel load

is 13365 lbs. making the concentrated loads equal 19682 + 13365 = 33047 lbs. These loads are concentrated 7 ft. center to center upon a girder whose length is 28 ft. The weight of the girder will be assumed to be 130 lbs. per lin. ft., and its bending moment is 1/8 x 130 x 784 x 12 = 147200 lbs. The bending moment due to the concentrated loads is (49570 x 14 - 33047 x 7) 12 = 5551812 in. lbs. making a total bending moment of 5699000 in. lbs. The required section modulus then is 5699000 ÷ 16000 = 356.2. Therefore a riveted plate girder will be used having a web plate 26" x 5/16", four flange angles 5" x $3\frac{1}{2}$" x $\frac{1}{2}$", and two plates 12" x $\frac{1}{2}$" whose total section modulus is 370.7 in.^{3} and whose weight is 122.8 lbs. per lin. ft.

Girder 9 - 10

The concentrated load upon the center of the 14 ft. girder consists of the reaction due to the 26 ft. joists and its load, which equals

$\frac{1}{2}$ x 39364 = 19682 lbs. plus the reaction due to the 18 ft. joists and its load, which is $\frac{1}{2}$ x 26730 = 13365 lbs. making a total load of 33047 lbs. The bending moment due to the concentrated load is $\frac{1}{4}$ x 33047 x 14 x 12 = 1388000 in. lbs. and the bending moment of an assumed 60 lb. girder is 1/8 x 60 x 196 x 12 = 17640 in. lbs. making a total bending moment of 1405640 lbs. The req. section modulus then will be 1405640 ÷ 16000 = 88. An 18" - 55# beam whose section modulus is 88.4 in.^{3} will be used.

Girder 15 - 16

The reaction due to the joists is 39364 lbs. and is concentrated at the center of the 14 ft. span. The bending moment due to this concentrated load is $\frac{1}{4}$ x 39364 x 14 x 12 = 1653290 in. lbs. and the bending moment of an assumed 60 lb. beam is 1/8 x 60 x 196 x 12 = 17640 lbs. making a total bending moment of

1670930 lbs. The required section modulus will then be 1670930 ÷ 16000 = 104.4. A 21" – 57.5 lb. beam having a section modulus of 116.9 in.^{3} will be used.

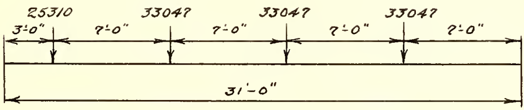
Girder 24 – 25

The reaction due to the joists and its panel load is 39364 lbs. The weight of the wall partition is 15 x 1 x 25 = 375 lbs. per lin. ft. and the assumed weight of the girder will be 60 lbs. per lin. ft., thereby giving a bending moment due to the uniform load as 1/8 x 435 x 196 x 12 = 128000 in. lbs. The bending moment due to the center concentrated load is $\frac{1}{4}$ x 39364 x 14 x 12 = 1653290 in.lbs., making a total bending moment of 1781290 lbs. The required section modulus is 1781290 ÷ 16000 = 111.4. Therefore a 21" – 57.5# beam having a section modulus of 116.9 in.^{3} will be used.

Girder 11 – 12

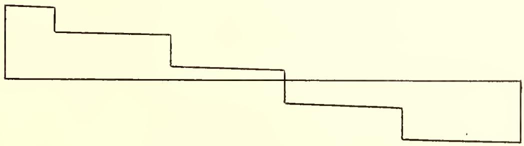
The reaction due to an 18" – 65# beam and

its panel load is $\frac{1}{2}$ x 39364 = 19682 lbs., and that due to a 15" - 36# beam and its panel load is $\frac{1}{2}$ x 26730 = 13365 lbs., making a concentrated load of 33047 lbs. The reaction of the spandrel girder 12 - 13 is 25310 lbs. These loads are concentrated 7 ft. and 3 ft. respectively upon a girder whose span is 31' as shown in the figure.



The maximum reaction due to the concentrated loads is 33047 (7 - 14 - 21) - 25310 x 28 - 31 = 67634 lbs. Assume a girder whose weight is 130 lbs. per lin. ft. giving a total weight of 4030 lbs.

By the construction of a shear - diagram as shown



it is found that the bending moment is a maximum 14 ft. from one end. Therefore the bending moment due to the concentrated load is (67634 x 17 - 25310 x 14 - 33047 x 7) 12 = 6,770,000 in. lbs. and that due to the uniform weight of the girder is (2015 x 17 - 130 x 17 x 8.5) 12 = 185,640 in. lbs. thereby making a total bending moment of 6,956,000 in. lbs. The required section modulus then will be 6956000 ÷ 16000 = 409.8. A riveted plate girder will be used having a web plate 26" x 3/8", four flange angles 5" x 3½" x ½", and two plates 12" x 5/8", whose total section modulus is 415.2 in.^{3}.

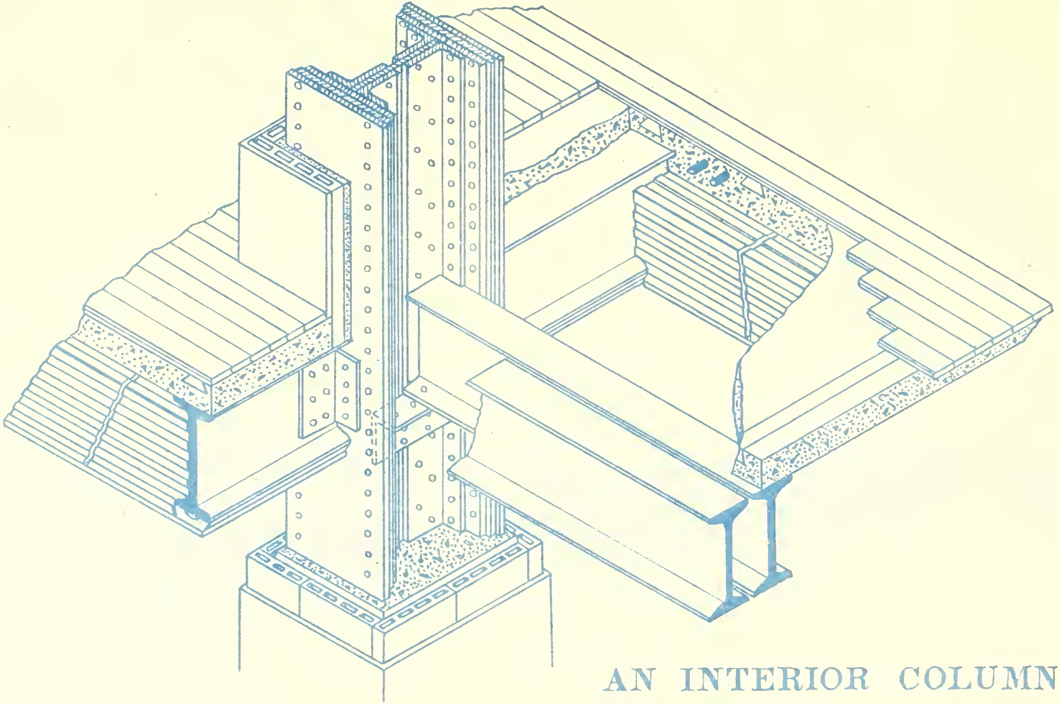
COLUMNS.

The columns will be composed of plates and angles forming an "H" shaped cross section and will be encased in one or more of fireproof material consisting of about three to four inches of brick, hollow terra cotta, concrete, burnt clay tiles, or of a combination of any two of these materials, provided that their combined thickness is not less than three inches.

For the design of the steel columns encased in one of the required fireproofing materials, which extends at least three inches beyond the outer edge of the steel and where the steel is calculated to carry the entire live and dead loads, the allowable stress per square inch will be determined by the following formula:

$$18000 - 70\,\frac{L}{r}$$

but shall not exceed 16000 lbs. The length in



AN INTERIOR COLUMN

inches equals L and R equals least radius of gyration in inches.

The length of the columns will be two stories.

Eccentric Loading.

Stresses due to eccentric loading shall be provided for in columns having an eccentric loading by the following formula:

$$f = P + \frac{Pea}{r^2}$$

where P is equal to one-half of the live load per bay, e, is the eccentric distance which is assumed as 7", a, the distance of the neutral line from the outer edge, which is assumed as 7", and r, the radius of gyration which is assumed as 5 inches.

An average eccentric loading will be taken in the design of the columns, which will be the live load from a second story bay.

Live load per bay 26 x 28 $= 61880\#$

Therefore P. will $= 61880 \div 2 = 30940$

$$f = 30940 + \frac{30940 \times 7 \times 7}{25}$$

$$= 91580$$

Therefore the eccentric effect will equal

$91580 - 61880 = 29700$ lbs.

Column Design.

An example of a column design for a section of column 14 is as follows:

Load sustained upon the column section 16 feet long is 234,206 lbs.

From the hand book a plate and angle columns is assumed, whose least radius of gyration is 2.61.

The allowable fiber stress, $S = 18000 - 70\,\frac{16 \times 12}{2.61} = 12{,}750$ lbs. per sq. in.

Required area, $A = 234{,}206 \div 12{,}750$

$= 18.3$ sq. in.

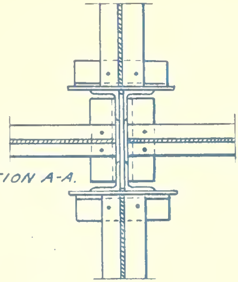
Therefore a column section having a web plate 10" x 3/8", four angles 6" x 4" x 7/16" and an area of 20.47 sq. in.

will be used.

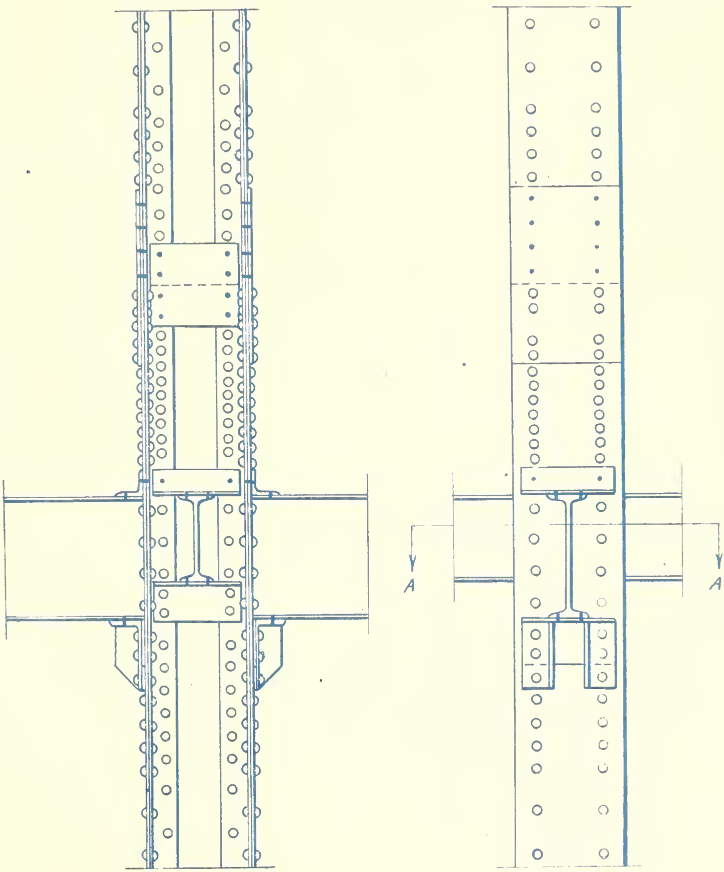
Strength of column will equal

$$20.47 \times 12,750 = 259,380 \text{ lbs.}$$

TYPICAL COLUMN SPLICE.



SECTION A-A.



Schedule for Column 2.

	Roof	2nd fl.	1st fl.	Basem't.
Floor L. L.	6300	25200	25200	
Floor D. L.	25100	29300	31070	
Column & covering		3060	3060	5100
Wall	36520	36520	36520	
Total for story	67920	94080	95850	5100
Accumulated total	67920	162000	257850	262950
Eccentric Effect	8740	29700	29700	
Total	76660	191700	287550	262950
Section of Column	s = 11750 A = 17.87	Web = 10"x3/8" 4 s = 5"x3½"x7/16"	s = 12925 A = 22.75	Web = 10"x3/8" 4 s = 6"x4"x½"

Schedule for Column 11.

	Roof	2nd fl.	1st fl.	Basem't.
Floor L. L.	15400	52360	49280	
Floor D. L.	50096	78964	74228	
Column & covering		6120	6120	5100
Total for story	65496	137444	129628	5100
Accumulated total	65496	202940	332568	337668
Total	65496	202940	332568	337668
Section of Column	s = 11750 A = 17.87	Web = 10"x3/8" 4 s = 5"x3½"x7/16"	s = 12950 A = 26.24	Web = 1-0"x½" 4 s = 6"x4"x9/16"

Schedule for Column 13.

	Roof	2nd fl.	1st fl.	Basem't.
Floor L. L.	9100	30940	28620	
Floor D. L.	29595	44408	44288	
Column & covering		3060	3060	5100
Wall	29960	29960	29960	
Total for story	68655	108368	105929	5100
Accumulated total	68655	176736	282664	287764
Eccentric Effect	8740	29700	29700	
Total	77395	206436	312364	287760
Section of Column	s = 11750 A = 17.87	Web = 10"x7/8" 4 s = 5"x3½"x7/16"	s = 12770 A = 25	Web = 12"x½" 4 s = 6"x4"x½"

Schedule for Column 14.

	Roof	2nd fl.	1st fl.	Basem't.
Floor L. L.	18200	61880	57240	
Floor D. L.	59190	88816	88576	
Column & covering		6120	6120	5100
Total for story	77390	156816	151936	5100
Accumulated total	77390	234206	386142	391242
Total	77390	234206	386142	391242
Section of Column	s = 12800 A = 20.47	Web = 10"x3/8" 4 s = 6"x4"x7/16"	s = 13023 A = 31.60	Web = 12"x$\frac{1}{2}$" 4 s = 6"x4"x11/16"

Schedule for Column 21.

	Roof	2nd fl.	1st fl.	Basem't.
Floor L. L.	9700	17884	21120	
Floor D. L.	50857	44408	44288	
Column & covering		6120	6120	5100
Total for story	60557	68412	71528	5100
Accumulated total	60557	128670	200500	205600
Total	60557	128670	200500	205600
Section of Column	s = 9850 A = 13.67	Web = 10"x3/8" 4 s = 4"x3"x3/8"	s = 11750 A = 17.87	Web = 10"x7/8" 4 s = 5"x3½"x7/16"

Schedule for Column 24.

	Roof	2nd fl.	1st fl.	Basem't.
Floor L. L.	9700	240550	21120	
Floor D. L.	128816	33324	44288	
Column & covering		6120	6120	5100
Total for story	138516	279994	71528	5100
Accumulated total	138516	418510	490038	495138
Total	138516	418510	490038	495138
Section of Column	s = 13058 A = 33.76	Web = 12"x$\frac{1}{2}$" 4 s = 6"x4"x$\frac{3}{4}$"	s = 13830 A = 37.50	Web = 12"x3/8" 4 s = 6"x4"x$\frac{1}{2}$" 2 Pls = 14"x$\frac{1}{2}$"

GRILLAGE FOUNDATIONS.

In the design of the grillage foundations for columns, provision has been made for the uniform distribution of the load over the footing by means of steel beams and concrete, which will be more economical than deep excavations. For medium loads, two tiers of beams will be necessary, while for light loads one tier of beams may suffice.

The lower tier of beams will be imbedded in concrete of sufficient area for the uniform distribution of the load over the footing. Thethickness of the concrete below the beams and about the sides and ends is usually made about 9 in. and the covering is made not less than 4 in.

The clear distance between the beam flanges in each tier shall not be made less than 2½ in. nor more than three times the flange width. Separators made of gas pipe

are usually used for the top tier to prevent spreading of the beams.

The following design of grillage foundation is that for column 24. The grillage foundations for other columns, the concrete area and the steel beams will vary in proportion to the respective loads over the footing.

Allowed bearing capacity = 4000# / sq. ft.

Load on column = 495,140#

The column consists of one web plate 12" x 3/8", four flange angles, 6" x 4" x ½" and two flange plates 14" x ½", whose outside dimensions are 13½" x 14".

Required area of footing equals

$495,140 \div 4,000 = 123.8$ sq. ft.

Use area 14' - 0" x 9' - 0" = 126 sq. ft.

It is assumed that the column base is 3' - 0" square and that 9" is allowed for concrete on the sides and ends of the beams, thereby making the dimensions of the steel grillage 12' - 6" x 7' - 6".

Rolled steel slab.

Thickness required, $t = \sqrt{\dfrac{3 \times 495{,}140 \times 22}{64000 \times 36}}$

$= 3.76$

Use a 3-7/8" plate.

Bottom tier - $L = 12.5$ ft.; $a = 3.0$ ft.

Required total section modulus,

$$S = \frac{3 \times 495140 \times 9.5}{32000} = 441 \text{ in.}^3.$$

Use 10 - 15" - 36# beams. Total section modulus $= 10 \times 54 = 540$ in.^{3}.

Average shear =

$$\frac{495140}{12.5} \times \frac{9.5}{2} \times \frac{1}{10 \times 15 \times 0.289}$$

$= 4373$ lbs. per sq. in.

Average buckling stress =

$$\frac{495140}{10 \times 43.5 \times 0.289} = 3930 \text{ lbs. / sq. in.}$$

Top tier - $L = 7.5$ ft.; $a = 3.0$ ft.

Required total section modulus,

$$S = \frac{3 \times 495140 \times 4.5}{32000} = 209 \text{ in.}^3.$$

Use 4 – 15" – 36# beams. Total section modulus = 4 x 54 = 216 in.^{3}.

Average shear =

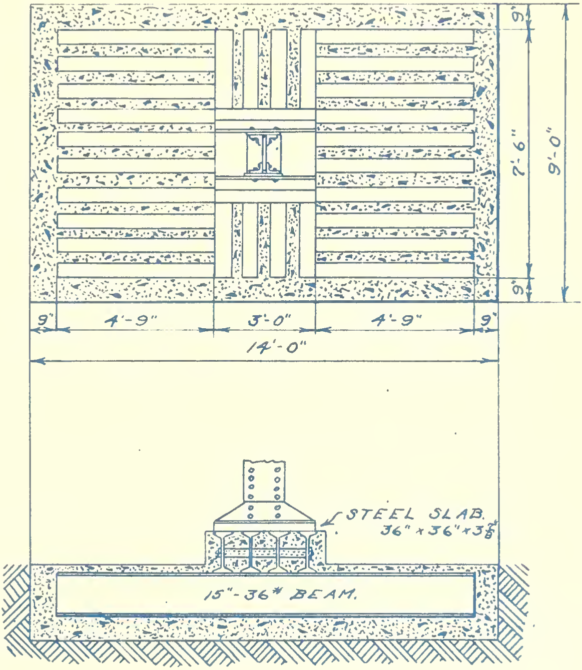
$$\frac{495140}{7.5} \times \frac{4.5}{2} \times \frac{1}{4 \times 15 \times 0.289} = 8655$$

lbs. per sq. in.

Average buckling stress =

$$\frac{495140}{4 \times 43.5 \times 0.289} = 9843 \text{ lbs. / sq. in.}$$

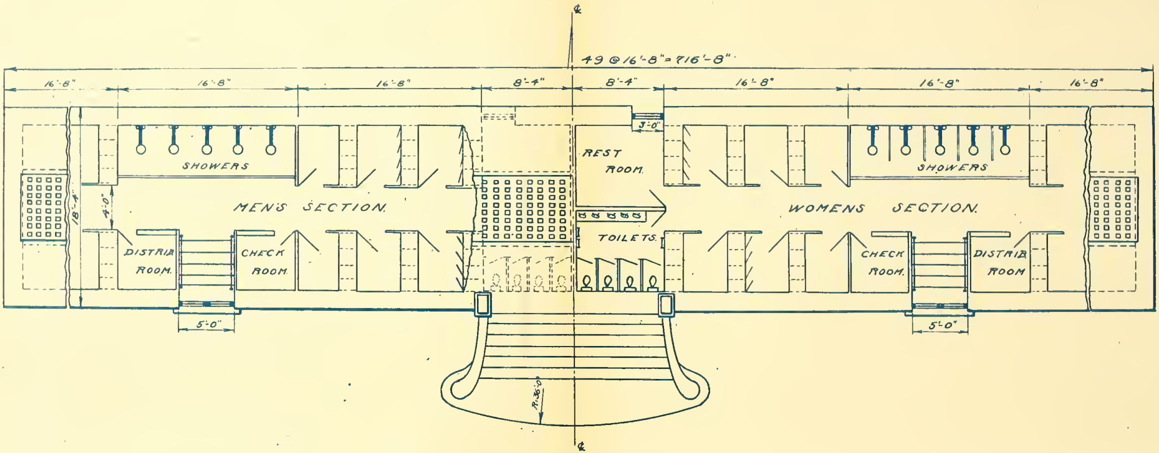
GRILLAGE FOUNDATION.
FOR COLUMN 24



SCALE ¾" = 1'-0"

PART II.

DESIGN OF REINFORCED CONCRETE CONSTRUCTIONS.



PLAN OF LOCKERS, SHOWERS, ETC.
SCALE $\frac{1}{8}$" = 1'-0"

NOTE.-
LOCKER ROOMS, 5'-6" × 4'-0"
LOCKERS, 5'-6" × 1'-6"

DESIGN OF THE CANTILEVER REINFORCED CONCRETE RETAINING WALL

The retaining wall is designed to restrain an earth bank $3\frac{1}{2}$ ft. high, a 6 in. concrete walk, and balustrade; assuming the foundation to be 3 ft. below the natural surface. The width on top, exclusive of the projection of the coping, is to be 18 inches, and the faces of the walls are vertical. Thickness of the footing at each edge is assumed to be 12 inches and 18 inches at the stem. Then the total height of the wall will be 7 feet. The length of the footing will be assumed to be tentatively 4 ft. 7 in.

It will be assumed that the surface of the back-filling is level; that the ratio of the horizontal pressure to the vertical pressure is equal to 1/3; that the weight of a cu. ft. of earth is 100 lbs.; that the weight of concrete is 150 lbs. per cu. ft.; and that the live load is 100 lbs. per sq. ft.

WEIGHTS

Weight of Concrete

Post (3' x 2' x 1.5')	= 3 x 1.5 x 150	= 675
Pavement 6 in.	= 2 x 0.5 x 150	= 150
Stem	= 7 x 1.58 x 150	= 1659
Heel	= (2x1+1x0.5) x 150	= 375
Toe	= (1x1+0.5x0.5) x 150	= 188
	Total lbs......	3047

Weight of Earth

Earth = (4 x 2 + 1 x 0.5) x 100 = 850#

Weight of Live Load

Live Load = 2 x 100 = 200#

Total Weight

Concrete, Earth and Live Load = 4097#

Center of Gravity

The center of gravity is equal to the sum of the area moments divided by the sum of the areas.

$$\frac{11.5\times0.979+2.5\times1.083+4.5\times2.75+11.06\times2.792+1.25\times4.03}{30.81}$$

$= 2.023$ or $2' - \frac{1}{4}"$

Horizontal Pressure.

$$P = \frac{wz}{3} \times \frac{z}{2} = \frac{wz^2}{6}$$

$$= \frac{100 \times 7^2}{6} = 817 \text{ lbs.}$$

Stability against Sliding.

Coefficient of friction may be taken as 0.5; and the frictional resistance to sliding will then be 4097 x 0.5 or 20485 lbs. Under this condition the wall is safe, and therefore a projection on the under side is not necessary.

Bending Moment.

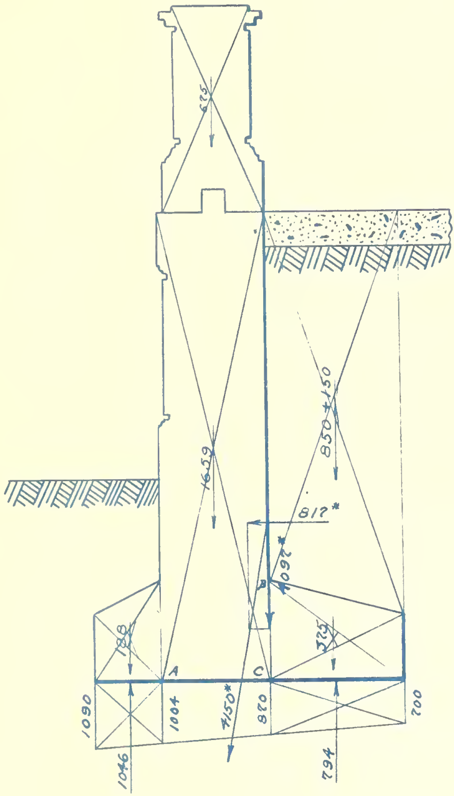
$$M = w\frac{Y^2}{6}\left(\frac{Y}{3}\right) = u\frac{Y^3}{18}$$

$$= 100 \times (5.5^3 \div 18) = 725\#$$

Stability against Overturning.

The stability against overturning is found by drawing the actual cross section of the wall and determining graphically where the

CROSS-SECTION
OF
RETAINING WALL.



SCALE ½"=1'-0" & 1"=5000* & 2000*

resultant cuts the middle third as in the figure. The resultant was found to fall within the middle third and therefore the cross section of the wall is satisfactory.

Pressure on the Soil.

To determine the pressure on the soil,

$$P = \frac{W}{l} \pm \frac{6Wd}{l^2}$$

$$= \frac{4097}{4.583} \pm \frac{6\ (4097)\ (0.167)}{21}$$

$$= 894 \text{ lbs.} \pm 195 \text{ lbs.}$$

Pressure on soil at toe is 1089 lbs. per sq. ft.

Pressure on soil at heel is 699 lbs. per sq. ft.

Reinforcement in the Stem.

The bending moment of a section of the stem 1 ft. long about any point in the plane of the footing is

$M = 817 \times 1.33 \times 12 = 13040$ in. lbs.

The moment of the tension in the steel, T,

is Tjd; or M = Tjd. Ordinarily j is taken as 7/8 and T then is,

$$T = M \div jd = 13040 \div 14$$
$$= 932 \text{ lbs. / lin. ft. of wall}$$

If $f_s = 16{,}000$, the required area of steel / lin. ft. of wall will be,

$$a_s = T \div f_s = 932 \div 16{,}000$$
$$= 0.0583 \text{ sq. in.}$$

Use $\frac{1}{4}$" ∅ rods and space 6" C. to C.

The fiber stress in the concrete,

$$f_c = 2M \div Kjbd^2$$
$$= (2 \times 13040) \div (0.375 \times 0.875 \times 12 \times 16^2)$$
$$= 29.11 \text{ lbs. per sq. in.}$$

The unit shear,

$$v = \frac{V}{bjd} = 817 \div (12 \times 0.875 \times 16)$$
$$= 4.84 \text{ lbs. per sq. in.}$$

The unit bond,

$$u = \frac{V}{\Sigma o jd} = \frac{817}{1.57 \times 0.875 \times 16}$$
$$= 37 \text{ lbs. per sq. in.}$$

Rods should be embedded about forty diameters below the base of the stem and form a complete loop about a horizontal rod.

Reinforcement in Front Part of Footing.

This position of the footing acts as a cantilever to transmit pressure to the soil; and therefore is reinforced on the lower side. The unit pressure at the toe is 1089 lbs. per sq. ft.; and at the heel is 699 lbs. per sq. ft.

Pressure at A

$$= 699 + (1089 - 699) \times \frac{3.583}{4.583}$$

$$= 1004 \text{ lbs. per sq. ft.}$$

The center of gravity of the pressure from A is 0.521 ft. and the moment about A is :

$$M = \left[(1089 + 699) \div 2\right] \times 1 \times 0.521 \times 12$$

$$= 5590 \text{ in. lbs.}$$

$$T = M \div jd = 5590 \div (0.875 \times 16)$$

$$= 400 \text{ lbs. per lin. ft.}$$

$$a_s = T \div f_s = 400 \div 16000 = 0.025 \text{ sq. in.}$$

Theoretically this shows that reinforcement is not necessary, but it would be advisable if one rod was used.

Reinforcement in Rear Part of Footing.

This portion of the footing will also act as a cantilever. It will carry the uniform downward pressure upon its upper face, in addition to its own weight; also a uniformly varying upward pressure on its lower face. The moment of the downward pressure at B is 1050 x 1.042 or 1094 ft. lbs. The weight of the footing is (2 x 1 + 1 x 0.5) x 150 or 375 lbs.; and its moment about B is 375 x 1 = 375 ft. lbs. The total downward moment then is 1470 ft. lbs.

The upward pressure at the heel is 699 lbs. / sq. ft., and the pressure at C is

$$= 699 + (1089 - 699) \times \frac{2}{4.583}$$

$$= 870 \text{ lbs. per sq. ft.}$$

The upward moment M is

$$870 \times 2 \times 0.96 = 1690 \text{ ft. lbs.}$$

Therefore the net downward moment at B is

$$1690 - 1470 = 220 \text{ ft. lbs.} = 2640 \text{ in. lbs.}$$

Required area of steel is

$$T = M \div jd = 1470 \div (0.875 \times 14)$$

$$= 120 \text{ lbs. per lin. ft.}$$

$$a_s = T \div f_s = 120 \div 16000$$

$$= 0.007 \text{ sq. in.}$$

Therefore reinforcement is not necessary, but several rods will be used to provide for the differences in the bearing power of the soil.

Temperature Reinforcement.

To prevent unsightly temperature cracks, the walls will be provided with longitudinal reinforcement to resist temperature stresses and contraction joints to localize the cracks. The amount to use may vary from 0.2 of 1% to 0.4 of 1%, based on the cross section of the concrete.

Steel in the front face of wall will be o.2 of 1% and the rods will be spaced with 6 in. centers. Cross section area of stem is

$$1.58 \times 5.5 \times 12 \text{ or } 104.3 \text{ sq. in.}$$

Required area of steel

$$a_s = 104.3 \times 0.002$$

$$= 0.2084 \text{ sq. in. per lin. ft.}$$

Use 3/8" ø rods.

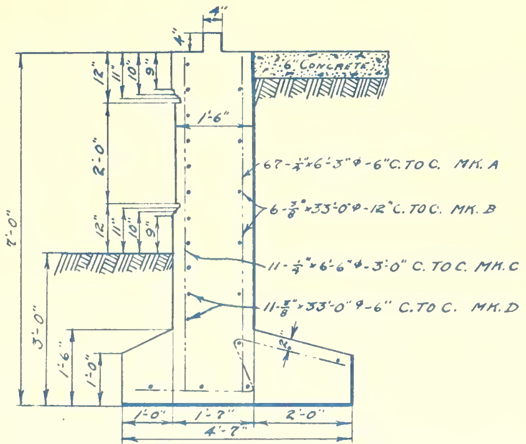
Steel in the rear face of wall will be 0.1 of 1% and the rods will be spaced with 12 in. centers. Cross section area of stem is 104.3 sq. in.

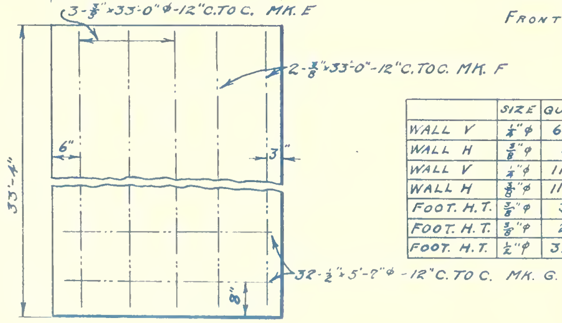
Required area of steel

$$a_s = 104.3 \times 0.001$$

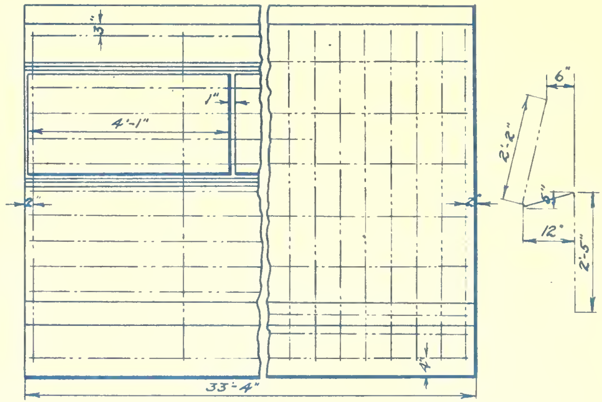
$$= 0.1043 \text{ sq. in. per lin. ft.}$$

Use 3/8" ø rods.





PLAN OF FOOTING.



FRONT ELEVATION. REAR ELEVATION.

	SIZE	QUAN.	LTH.	MK.
WALL V	¼"φ	67	6'-9"	A
WALL H	⅜"φ	6	33'-0"	B
WALL V	¼"φ	11	6'-6"	C
WALL H	⅜"φ	11	33'-0"	D
FOOT. H.T.	⅜"φ	3	33'-0"	E
FOOT. H.T.	⅜"φ	2	33'-0"	F
FOOT. H.T.	½"φ	32	5'-7"	G

REINFORCED CONCRETE
RETAINING WALL.

NOTE:-
EXPANSION JOINT EVERY
33'-4"

SCALE ⅜" = 1'-0"

DESIGN OF PROMENADE.

Slab.

In the following design the working stresses recommended by the Joint Committee for a 2000 lb. concrete will be used; namely

$f_s = 16000, \quad f_c = 650, \quad n = 15$

and the following values,

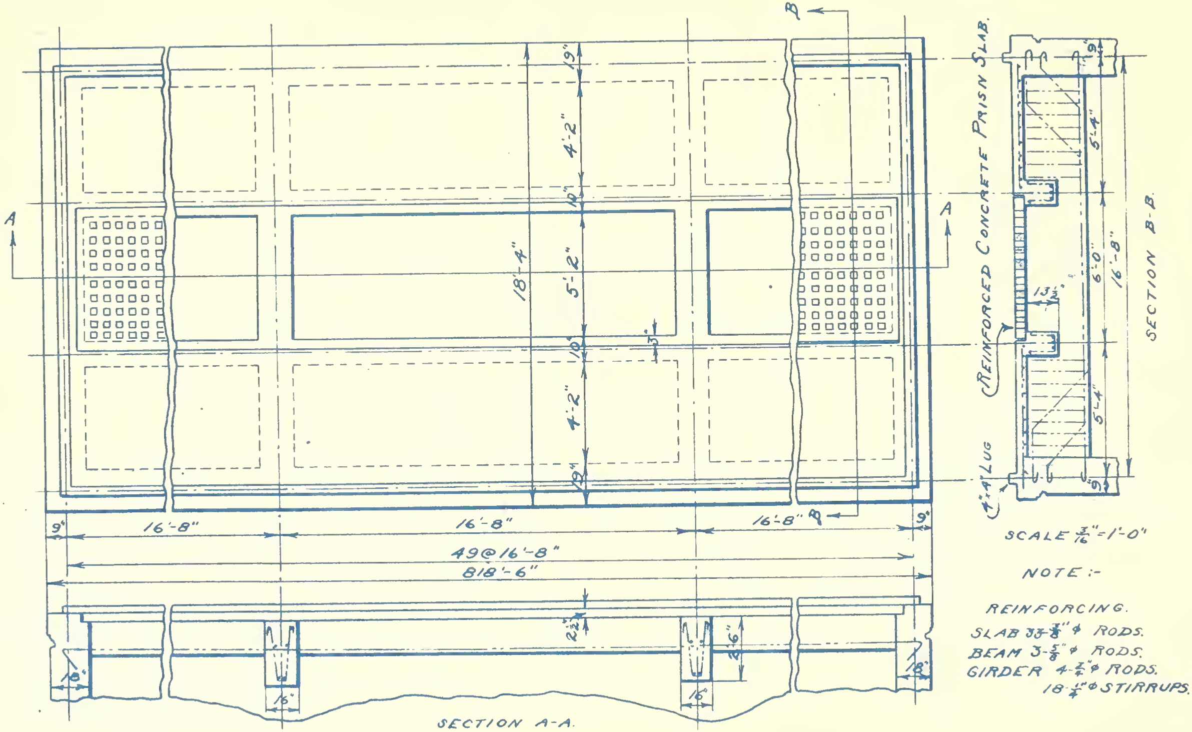
$p = 0.0077, \quad k = 0.378, \quad j = 0.874$

The design will be in accordance with the requirements of the Revised Building Ordinances of the City of Chicago, Seventh Edition, March, 1916.

The slab is to span 5.33 ft., to carry its own dead weight, and to carry a live load of 100 lbs. per sq. ft. Slab is to be reinforced in only one direction.

Assuming a 3 in. slab and a width of 12 in. the weight of slab per ft.

$$w = \frac{bd \times 12}{144 \times 12} \times 150 = 1.04 \text{ bd.}$$



PLAN OF PROMENADE.

Therefore the dead load is 3 x 12 x 1.04 or 38 lbs., making a total loading of 138 lbs. per sq. ft.

$$M = \frac{12}{10} wl^2 = 138 \times 5.33^2 \times 1.2$$

$$= 4700 \text{ in. lbs.}$$

Safe moment of resistance

$$M = pf_s jbd^2 = 1290d^2$$

$$d^2 = 4700 \div 1290 = 3.64 \text{ or } d = 2 \text{ in.}$$

Hool: Table 6

Depth to steel = $2\frac{1}{4}$ in.

Depth below steel = 3/4 in.

Area of steel per ft. of slab = 0.208 sq. in.

Use 3/8" ø rods and space with 6 in. centers.

$$v_o = \frac{V}{bd} = \frac{138 \times 2.67}{12 \times 3} = 10.2 \text{ lbs./ sq. in.}$$

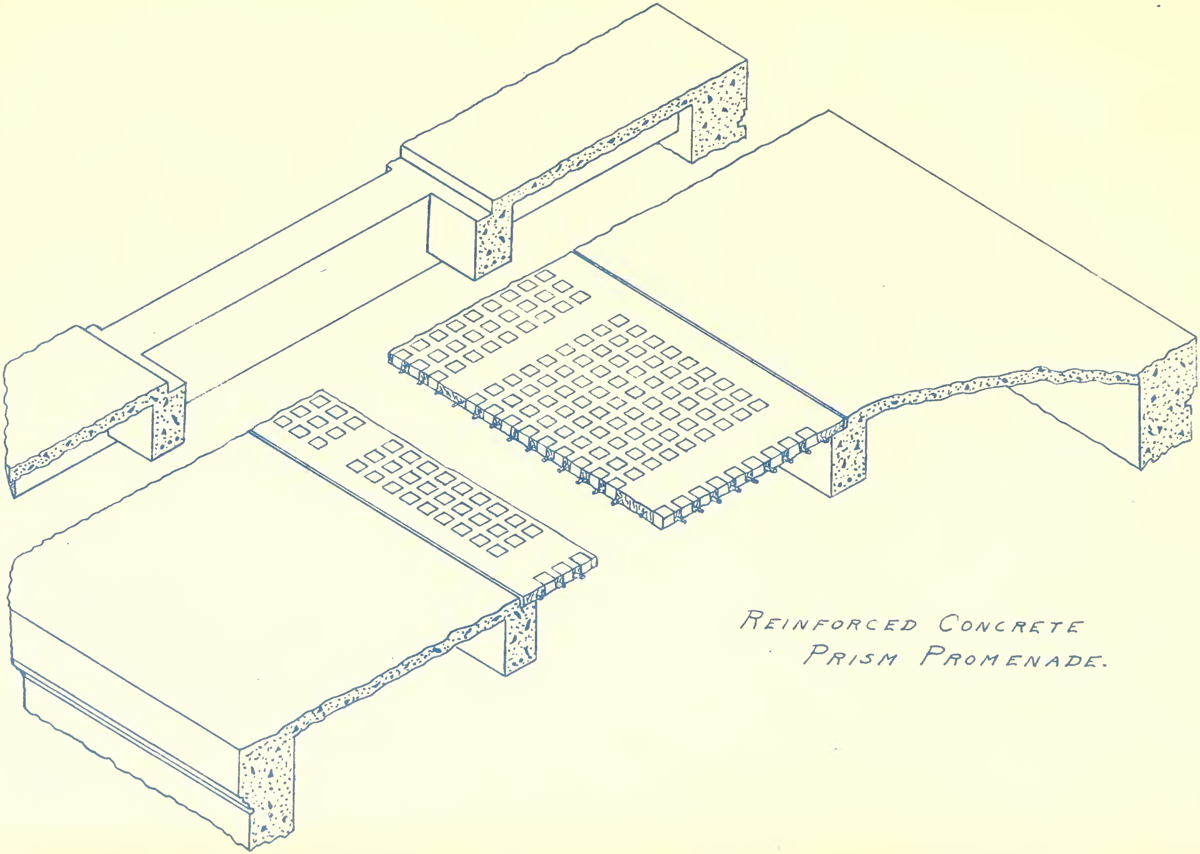
Allowed shear

$$vbd = 40 \times 12 \times 3 = 1440 \text{ lbs.}$$

Required spacing of rods is,

$$\frac{0.110}{0.227} \times 12 = 6.4 \text{ in. on centers.}$$

The slab is to be reinforced against negative



Reinforced Concrete
Prism Promenade.

moment at the supports, and to be reinforced transversely in order to prevent shrinkage and temperature cracks. The amount of steel will be 0.2 of 1%, based on the cross section, which is 64 x 3 or 192 sq. in.

Required area of steel

$$a_s = 192 \times 0.002$$

$$= 0.384 \text{ sq. in. / lin. ft.}$$

Use ½" ø rods and spaoe with 6 in. centers.

The center portion of the promenade is to be constructed with a reinforced concrete prism sidewalk having a width of 6 ft.

Rectangular Beam.

Beam is to span 16.67 ft. and to carry the live and dead load from the promenade (including weight of beam).

Weight per lin. ft. from slab is

$$138 \times 2.67 = 369 \text{ lbs.}$$

Weight per lin. ft. from prism walk is

$$138 \times 3 = 414 \text{ lbs.}$$

Assume weight of beam per lin. ft. as

167 lbs.

Total load per lin. ft. = 950 lbs.

Moment

$$M = \frac{12}{10} wl^2 = 950 \times 16.67^2 \times 1.2$$

$$= 316920 \text{ in. lbs.}$$

$$bd^2 = \frac{316920}{(0.0077)(16000)(0.874)} = 2934$$

Assume $b = 10$ in.

$$d^2 = \frac{2934}{10} = 294, \text{ or } d = 17 \text{ in.}$$

Thickness of slab is 3 in., therefore depth of the beam is 17 - 3 or 14 in. Allow $1\frac{1}{2}$" of concrete below reinforcement for fire-proofing, thus making the final depth $15\frac{1}{2}$ in.

Check for weight of the designed beam

$= 10 \times 15.5 \times 1.04 = 161$ lbs. / lin. ft.

The assumed and calculated weights are close enough so that the beam need not be re-designed.

Area of cross section

$$bd = 10 \times 17 = 170 \text{ sq. in.}$$

$$a_s = 170 \times 0.0077 = 1.309 \text{ sq. in.}$$

Use 3 - $\frac{3}{4}$" ø rods, whose area is 3×0.4418 or 1.3254 sq. in.

Spacing of these rods will be 3 in. C. to C.

The maximum bond

$$u = \frac{V}{\Sigma_0 jd} = \frac{950 \times 8.34}{3 \times 2.36 \times 0.874 \times 17}$$

$$= 75 \text{ lbs. per sq. in.}$$

Therefore the bond stress is satisfactory.

Reviewing the beam designed,

$$p = \frac{as}{bd} = \frac{1.3254}{10 \times 17} = 0.0078$$

$$k = \sqrt{2\,pn + (pn)^2} - pn = 0.38$$

$$j = 1 - 1/3k = 0.874$$

$$f_s = \frac{12\ wl^2}{a_s bdj} = \frac{950 \times 278 \times 12}{10 \times 1.325 \times 0.874 \times 17}$$

$$= 15846 \text{ lbs. per sq. in.}$$

$$f_c = \frac{2\ fs\ p}{k} = \frac{2 \times 15846 \times 0.0078}{0.38}$$

$$= 640 \text{ lbs. per sq. in.}$$

which is satisfactory.

Web reinforcement will be needed and the average unit shear

$$v_o = \frac{V}{bd} = \frac{950 \times 8.34}{10 \times 17} = 46 \text{ lbs. / sq. in.}$$

Allowable average shear is 35 lbs. per sq. in. Therefore we shall provide the web re-

inforcement by means of vertical U shaped stirrups bent at the upper end and $\frac{1}{4}$ in. round rods may be considered secure against slipping

Stirrups are unnecessary at a distance from the center of support equal to

$$x = \frac{1}{2} - \frac{v^1 bjd}{w}$$

$$= \frac{16.67}{2} - \frac{40 \times 10 \times 0.874 \times 17}{950} = 2 \text{ ft.}$$

The minimum spacing of stirrups will occur at the support and will be equal to

$$s = \frac{3 \; a_s \; f_s \; jd}{2 \; V}$$

$$= \frac{3 \times 2 \times 0.49 \times 16000 \times 0.874 \times 17}{2 \times 950 \times 8.34}$$

$$= 4.5 \text{ in.}$$

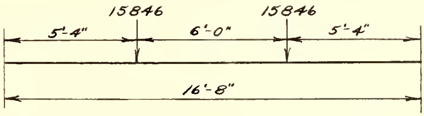
The stirrups are needed for such a short distance from each end of the beam that four stirrups spaced 6 in. C. to C. will suffice.

Rectangular Girder.

Girder is designed to span 16.67 ft. and to support the concentrated loads due to the

beams.

Shear due to one beam is 7923 x 2 or 15846 lbs. and are concentrated 5.33 ft. from each center line of support.



Assume weight of beam per lin. ft. as 500 lbs.

Moment due to weight of beam

$$M = \frac{12}{10} wl^2 = 1.2 \times 500 \times 8.342^2$$

$$= 417360 \text{ in.lbs.}$$

Moment due to concentrated load

$$M = (15846 \times 8.34 - 15846 \times 3)\ 12$$

$$= 1015414 \text{ in. lbs.}$$

Total bending moment

$$M = 1432774 \text{ in. lbs.}$$

$$bd^2 = \frac{1432774}{(0.0077)(16000)(0.874)} = 13270$$

Assume b = 16 in.

$$d^2 = \frac{13270}{16} = 830, \text{ or } d = 28.8 \text{ in.}$$

Check for weight of the designed girder

$$= 16 \times (2.88 + 1.5) \times 1.04$$

$$= 500 \text{ lbs. per lin. ft.}$$

Take b = 16 in., and d = 28.8 in.

Average unit shear

$$v_o = \frac{V}{bd} = \frac{20016}{16 \times 28.8} = 44 \text{ lbs. per aq. in.}$$

Allowable average shear is 35 lbs. per sq. in., and therefore web reinforcement is necessary.

$$a_s = bjd = 16 \times 0.0077 \times 28.8$$

$$= 3.54 \text{ sq. in.}$$

Use $8\frac{3}{4}$" round rods whose area equals $8 \times 0.4418 = 3.53$ sq. in. and space rods $1\frac{1}{2}$" C. to C.

The maximum bond for one rod

$$u = \frac{V}{\Sigma_o jd} = \frac{20016}{2.356 \times 0.874 \times 28.8}$$

$$= 337 \text{ lbs. per sq. in.}$$

Required number of horizontal rods

$$\Sigma_o = \frac{V}{ujd} = \frac{20016}{120 \times 0.874 \times 28.8} = 6.6$$

Therefore required number equals 6.6 ÷ 2.356 = 3 rods. Four rods will extend straight through the girder, and four rods will be bent up at each end.

The concrete will be found to take care of any diagonal tension between the concentrated loads.

Horizontal shear at the support is

$$\frac{V}{jd} = \frac{20016}{0.874 \times 28.8} = 788 \text{ lbs. / lin. in.}$$

and to the left of the adjacent concentrated load, it is

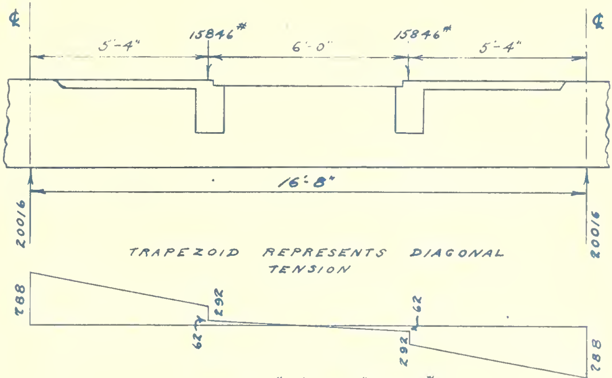
$$\frac{20016 - (500 \times 5.33)}{0.874 \times 28.8} = 292 \text{ lbs.}$$

per lin. in.

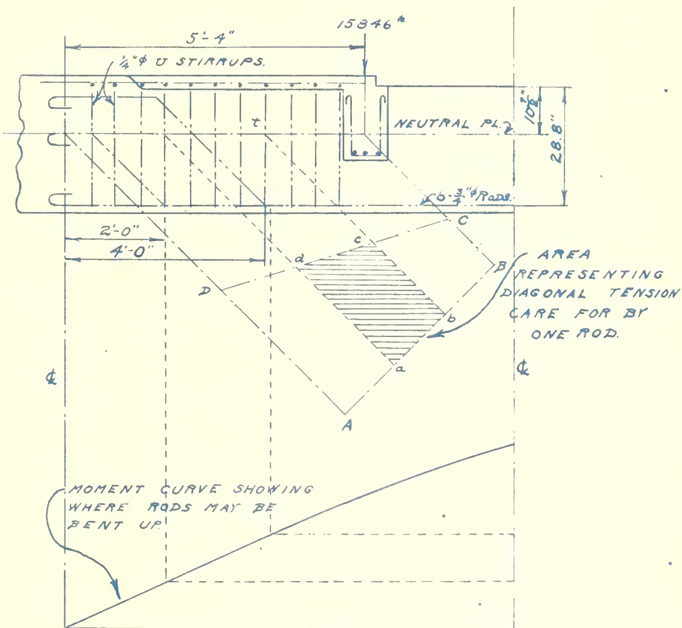
and below concentrated load it is

$$\frac{15864}{0.873 \times 28.8} = 62 \text{ lbs. / lin. in.}$$

Neutral plane of girder is kd = 0.378 x 28.8 = 10.9 in. from the top.



SCALE 1/4" = 1'-0" & 1/2" = 1000*



SCALE 3/8" = 1'-0" & 1" = 1,000,000*

manner.

An investigation is now made to see whether the tensile stresses in the bottom of the beam will permit the bending of the rods as required for diagonal tension. A bending moment curve in plotted to scale from the maximum moments at various sections and upon the center line the required areas of the rods are laid off, and horizontals are drawn as shown. Thus showing that rods may be bent up where these horizontals cut the curve.

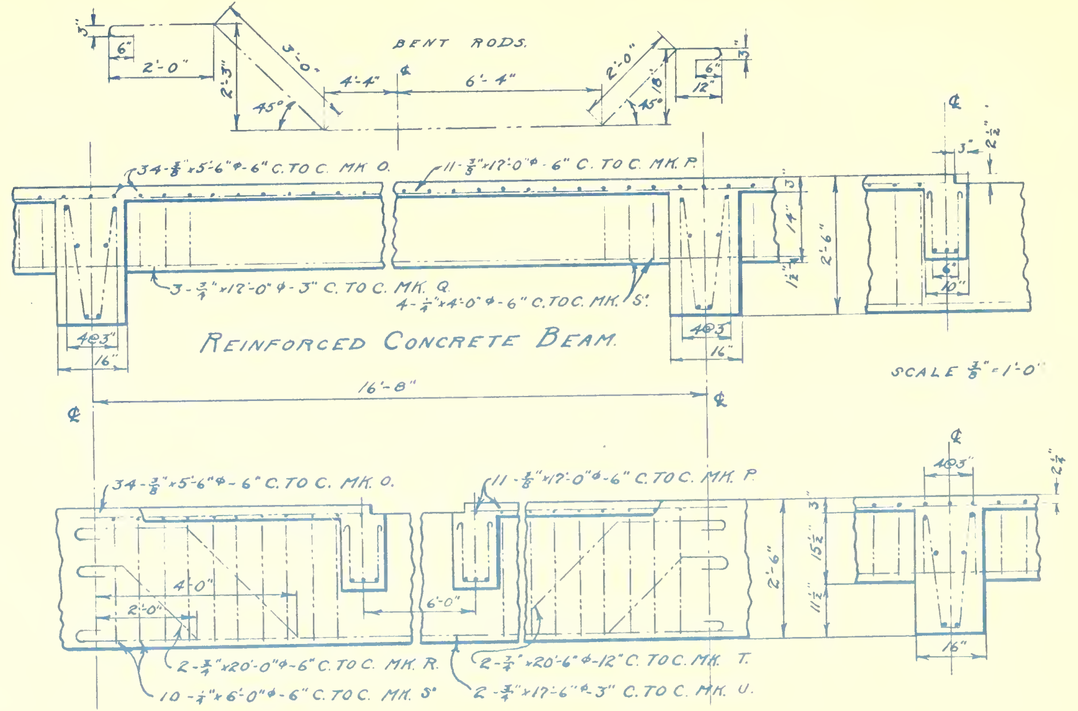
Stirrups will be provided to take diagonal tension between the point t in the figure where the line bc produced meets the neutral line and the adjacent load. It will be advisable in the design to place stirrups to the support.

The minimum spacing of stirrups

$$s = \frac{3\ a_s\ f_s\ jd}{2\ V}$$

$$= \frac{3 \times 2 \times 0.049 \times 16000 \times 0.874 \times 28.8}{2 \times 20016}$$

$$= 3 \text{ in.}$$



REINFORCED CONCRETE GIRDER.

DESIGN OF PROMENADE WALLS.

The promenade walls are designed to restrain an earth bank $7\frac{1}{2}$ ft. high and to support the promenade. The width on top, inclusive of the projection of the coping, will be 19 in., and the faces of the walls will be vertical. Total height of walls will be $11\frac{3}{4}$ ft., thus making the locker room height about 10 ft. The cross section of the walls will be similar to that of the retaining wall.

Reinforcing.

Reinforcement of steel will be of the same size and spacing as that used in the retaining wall, but using 3-3/8 in. round rods in the base of each footing in the longitude direction and $\frac{1}{2}$ in. round rods in the transverse direction.

Temperature Reinforcement.

The steel in the front face of the wall

will be 0.2 of 1%, based on the cross section of the concrete.

Cross section of the wall =

$1.58 \times 10.5 \times 12 = 200$ sq. in.

Required area of steel

$a_s = 200 \times 0.002 = 0.40$ sq. in. per lin. ft.

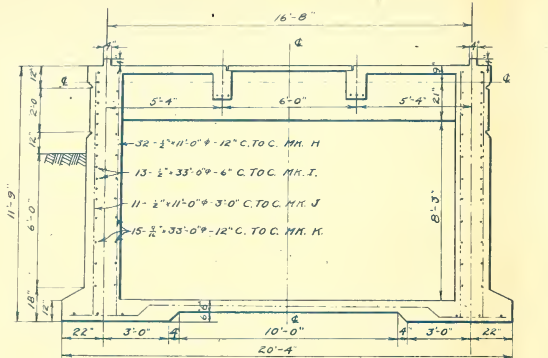
Use $\frac{1}{2}$ in. round rods and space 13 with 6 in. centers, and 4 with 12 in. centers.

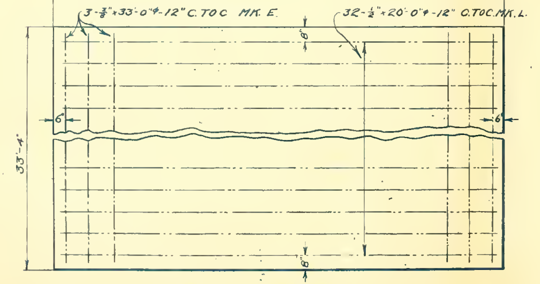
Cross section area of wall = 200 sq. in.

Required area of steel

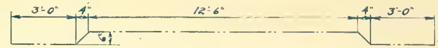
$a_s = 200 \times 0.001 = 0.20$ sq. in. per lin. ft.

Use 9/16 in. round rods.

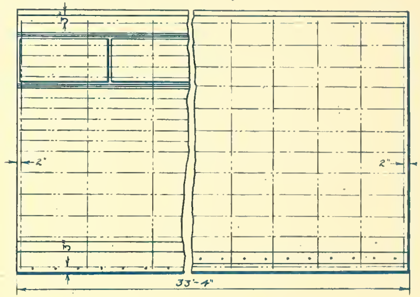




PLAN OF FOOTING.



BENT ROD



FRONT FACE. REAR FACE.

STEEL PER WALL, PER 33'-4"

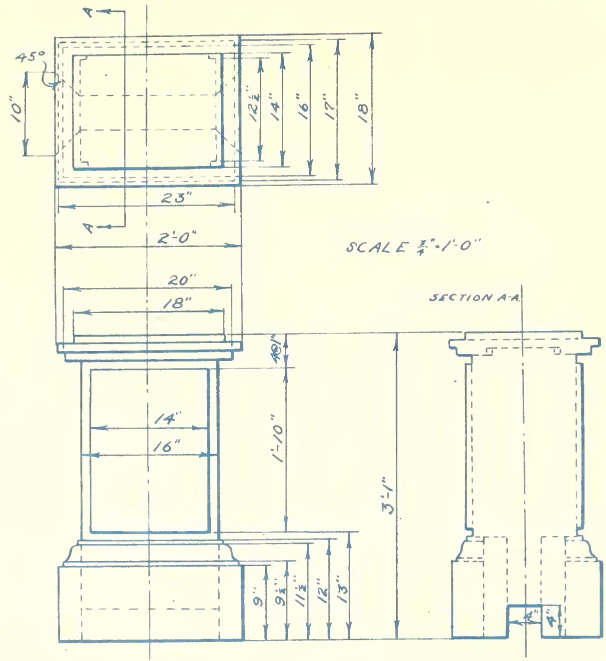
	SIZE	QUAN.	LTH.	MK.
WALL V	½"φ	32	11'-0"	H
WALL V	½"φ	11	11'-0"	J
WALL H	½"φ	13	33'-0"	I
WALL H	9/16"φ	15	33'-0"	K
FOOT H.T.	3/8"φ	3	33'-0"	E
FOOT H.T.	½"φ	32	20'-0"	L

REINFORCED CONCRETE PROMENADE WALLS.

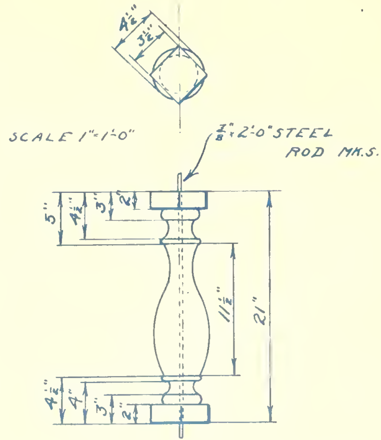
NOTE:-

EXPANSION JOINT EVERY 33'-4"

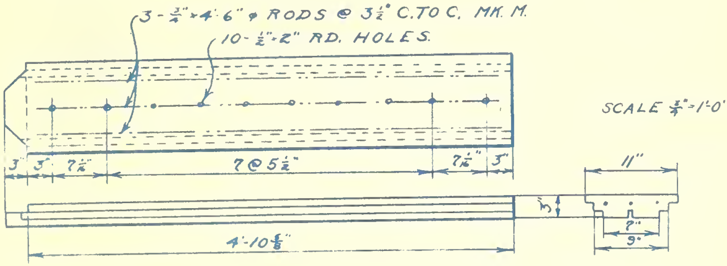
SCALE ¼" = 1'-0"



TYPICAL CONCRETE POST.

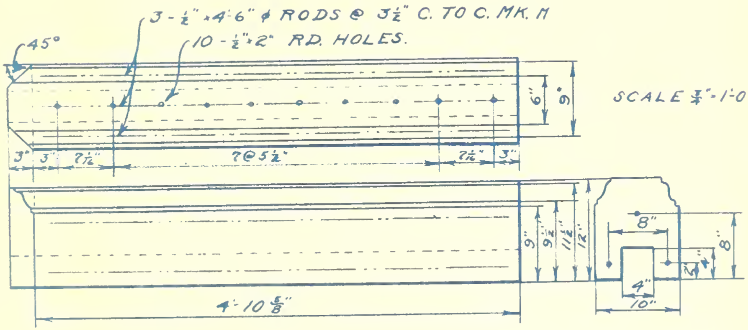


REINFORCED CONCRETE SPINDLE.

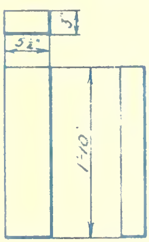


TOP OF BALUSTRADE.

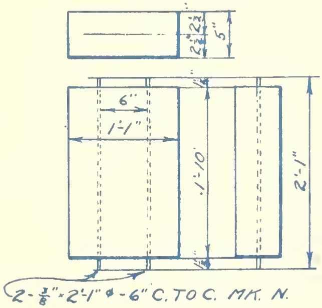
NOTE:-
FOR CENTER SECTIONS CUT ENDS SQ.



BASE OF BALUSTRADE.



END PIECE.



INTERMEDIATE POST.

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